Preface

Content
Interactive design aids in accordance to US codes ACI 318-11, AISC 14th edition and ASCE-7-10

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Chapter 1: Concrete Design

Design of Corbel as per ACI 318-11 Chapter 11

System
- Corbel Width, \( b = \) 14.0 in
- Corbel Height, \( h = \) 12.0 in
- Concrete Cover, \( c_o = \) 1.0 in
- Corbel Depth, \( d = \) \( h - c_o = 12.0 - 1.0 = 11.0 \) in
- Distance from Column Face to Vertical Load, \( a_v = \) 3.0 in

Load
- Ultimate Vertical Load, \( V_u = \) 88.8 kips
- Ultimate Horizontal Load, \( N_{uc} = \) 32.0 kips

Material Properties
- Concrete Strength, \( f'_c = \) 5000 psi
- Yield Strength of Reinforcement, \( f_y = \) 60000 psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = \) 0.75
- Modification Factor for Lightweight Concrete, \( \lambda = \) 1.00
- Friction Factor (According to Cl.11.6.4.3 of ACI318), \( \mu = 1.4 \times \lambda = 1.40 \)
Check Vertical Load Capacity

\[ V_{n1} = 0.2f'_c b d / 1000 = 154.0 \text{ Kips} \]
\[ V_{n2} = (480 + 0.08f'_c) b d / 1000 = 135.5 \text{ Kips} \]
\[ V_{n3} = 1600 b d / 1000 = 246.4 \text{ Kips} \]

Nominal Vertical Capacity (According to Cl.11.8.3.2.1 of ACI318),

\[ \phi V_n = \phi \times \text{MIN}(V_{n1}; V_{n2}; V_{n3}) = 101.6 \text{ Kips} \]

Vertical Load Capacity = \( \text{IF}(V_u > \phi V_n; \text{"Not Pass"}; \text{"Pass"}) = \text{Pass} \)

Determine Shear Friction Reinforcement (\( A_{vf} \))

Required Area of Reinforcement for Shear Friction (According to Cl.11.6.4.1 of ACI318),

\[ A_{vf} = \frac{V_u \times 1000}{(\phi \times f_y \times \mu)} = 1.41 \text{ in}^2 \]

Determine Direct Tension Reinforcement (\( A_n \))

Minimum Horizontal Force on Corbel, \( Nuc_{\text{min}} = 0.2 \times V_u = 17.8 \text{ Kips} \)

Horizontal Force on Corbel, \( Nuc_{\text{act}} = \text{MAX} (Nuc ; Nuc_{\text{min}}) = 32.0 \text{ kips} \)

Required Area of Reinforcement for Direct Tension (According to Cl.11.8.3.1 of ACI318),

\[ A_n = \frac{Nuc_{\text{act}} \times 1000}{(\phi \times f_y)} = 0.71 \text{ in}^2 \]

Determine Flexural Reinforcement (\( A_f \))

\[ M_u = V_u a_v + Nuc_{\text{act}}(h-d) = 298.4 \text{ kip} \times \text{in} \]

Required Area of Reinforcement for Flexural (According to Cl.11.8.3.3 of ACI318),

\[ A_f = \frac{M_u \times 1000}{(\phi \times f_y \times 0.9 \times d)} = 0.67 \text{ in}^2 \]

Determine Primary Tension Reinforcement (\( A_{sc} \))

Required Area of Reinforcement for Primary Tension (According to Cl.11.8.3.5 of ACI318),

\[ A_{sc} = \text{MAX} ((2/3 * A_{vf}) + A_n + A_f) = 1.65 \text{ in}^2 \]

Minimum Area of Reinforcement for Primary Tension (According to Cl.11.8.5 of ACI318),

\[ A_{sc_{\text{min}}} = 0.04 f'_c / f_y \times b \times d = 0.51 \text{ in}^2 \]

\[ A_{sc_{\text{Req}}} = \text{MAX} (A_{sc}; A_{sc_{\text{min}}}) = 1.65 \text{ in}^2 \]

Provided Reinforcement, Bar = \( \text{SEL("ACI/Bar"; Bar; ) = No.9} \)

Provided Area of Bar Reinforcement, \( A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar) = 1.00 \text{ in}^2} \)

Number of Provided Bars, \( n = 2 \)

Provided Area of Reinforcement, \( A_{sc_{\text{Prov}}} = n \times A_{sb} = 2.00 \text{ in}^2 \)

Check Validity = \( \text{IF}(A_{sc_{\text{Prov}} > A_{sc_{\text{Req}}}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \)
Determine Horizontal Reinforcement ($A_h$)

Required Area of Reinforcement for Horizontal Shear (According to Cl.11.8.4 of ACI318),

$$A_{h\_Req} = 0.5(A_{sc\_Prov}-A_n) = 0.65 \text{ in}^2$$

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.3

Provided Area of Bar Reinforcement, $A_{sb} = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 \text{ in}^2$

Number of Provided Bars, $n = 6$

Provided Area of Reinforcement, $A_{h\_Prov} = n \times A_{sb} = 0.66 \text{ in}^2$

Check Validity: IF($A_{h\_Prov} \geq A_{h\_Req}$; "Valid"; "Invalid") = Valid

Distribute in two-thirds of Effective Corbel Depth adjacent to $A_{sc}$

Design Summary

Area of Reinforcement for Primary Tension $A_{sc} = A_{sc\_Prov} = 2.00 \text{ in}^2$

Area of Reinforcement for Horizontal Shear, $A_h = A_{h\_Prov} = 0.66 \text{ in}^2$

Distribute in two-thirds of Effective Corbel Depth adjacent to $A_{sc}$
Design Precast Spandrel Beam for Combined Shear and Torsion as per ACI 318-11 Chapter 11

**System**
- Width of Beam, $b$ = 16.0 in
- Height of Beam, $h$ = 48.0 in
- Width of Beam Ledge, $b_L$ = 8.0 in
- Height of Beam Ledge, $h_L$ = 16.0 in
- Concrete Cover, $c_o$ = 2.50 in
- Concrete Cover to Center of Stirrup, $c_o'$ = 1.50 in
- Effective Depth of Beam, $d$ = $h - c_o$ = 45.50 in

**Load**
- Ultimate Bending Moment, $M_u$ = 1316.0 kip*ft
- Ultimate Torsional Moment, $T_u$ = 108.6 kip*ft
- Ultimate Shear Force, $V_u$ = 127.2 kips

**Material Properties**
- Concrete Strength, $f'_c$ = 5000 psi
- Yield Strength of Reinforcement, $f_y$ = 60000 psi
- Yield Strength of Stirrups Reinforcement, $f_yt$ = 60000 psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi_s$ = 0.75
- Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi_t$ = 0.90
- Modification Factor for Lightweight Concrete, $\lambda$ = 1.00
- Friction Factor (According to Cl.11.6.4.3 of ACI318), $\mu = 1.4 \times \lambda$ = 1.40
Determine Concrete Cracking Torque

Area Enclosed by Outside Perimeter of Spandrel Beam Including the Ledge,

\[ A_{cp} = b \cdot h + b_L \cdot h_L = 896 \text{ in}^2 \]

Outside Perimeter of Spandrel Beam Including the Ledge,

\[ P_{cp} = 2 \cdot (b + b_L + h) = 144 \text{ in} \]

Concrete Cracking Torque,

\[ T_{cr} = 4 \cdot \lambda \cdot \sqrt{f_c} \cdot \frac{A_{cp}^2}{P_{cp}} / 12000 = 131.4 \text{ kip*ft} \]

Torsional Moment should be: \( \text{IF}(T_u < \phi_s \cdot T_{cr}/4; \text{"Neglected"}; \text{"Checked"}) = \text{Checked} \)

Calculation of Torsion Reinforcement

Area Enclosed by Centerline of The Outermost Closed Transverse Torsional Reinforcement (According to Cl.11.5.3.6 of ACI318),

\[ A_{oh} = (h-2*co')*(b-2*co')+(b_L)*(h_L-2*co') = 689.0 \text{ in}^2 \]

\[ A_o = 0.85 \cdot A_{oh} = 585.6 \text{ in}^2 \]

Angle of Compression Diagonal Struts (According to 11.5.3.6 of ACI318),

\[ \Theta = 45^\circ \]

Required Area for Torsion Shear per Stirrups Spacing (According to Eq. 11-20, 21 of ACI318),

\[ A'_{vt} = \frac{T_u \cdot 12000}{2 \cdot \phi_s \cdot A_o \cdot f_{yt} \cdot (1 / \tan(\Theta))} = 0.025 \text{ in}^2 \text{ per in} \]

Calculation of Shear Reinforcement

Nominal Shear Strength Provided by Concrete (According to Eq.11-3 of ACI318),

\[ V_c = 2 \cdot \lambda \cdot \sqrt{f_c} \cdot \frac{b \cdot d}{1000} = 102.95 \text{ kips} \]

Nominal Shear Strength Provided by Reinforcement (According to Eq.11-2 of ACI318),

\[ V_s = V_u / \phi_s - V_c = 66.65 \text{ kips} \]

Required Area for Direct Shear per Stirrups Spacing (According to Eq. 11-1, 2 of ACI318),

\[ A'_{vs} = \frac{V_s \cdot 1000}{f_{yt} \cdot d} = 0.024 \text{ in}^2 \text{ per in} \]
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Precast Spandrel Beam for Combined Shear and Torsion

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Calculation of Combined Shear and Torsion Reinforcement

Total Required Area for Torsion & Shear per Stirrups Spacing (According to Cl.11.5.3.8 of ACI318),

\[ A'_{v} = A'_{vt} + \frac{A'_{vs}}{2} = 0.037 \text{ in}^2 \text{ per in per leg} \]

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.4
Provided Reinforcement, \( A_{sb} \) = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in^2

Required Stirrups Spacing, \( s_{req} \) = \( \frac{A_{sb}}{A'_{v}} \) = 5.41 in

Provided Stirrups Spacing, \( s_{prov} \) = 5.00 in

Check Validity = IF(\( s_{prov} \leq s_{req} \); "Valid"; "Invalid") = Valid

Perimeter of Stirrups, \( P_h \) = \( 2 \times (b - 2 \times c' + h - 2 \times c') + 2 \times b_L \) = 132.00 in

Maximum Stirrups Spacing Due to Torsion (According to Cl.11.6.6 of ACI318),
\( s_{max,t} = \min(P_h/8; 12) = 12.00 \text{ in} \)

Maximum Stirrups Spacing Due to Shear (According to Cl.11.4.5 of ACI318),
\( s_{max,v} = \min(d/2; 24) = 22.75 \text{ in} \)

Maximum Stirrups Spacing, \( s_{max} = \min(s_{max,t}; s_{max,v}) = 12.00 \text{ in} \)

Check Validity = IF(\( s_{prov} \leq s_{max} \); "Valid"; "Invalid") = Valid

Calculation of Longitudinal Torsion Reinforcement

Required Area of Longitudinal Torsion Reinforcement (According to Cl.11.5.3.7 of ACI318),

\[ A_{l,i} = \frac{A'_{vt} \times P_h \times f_{yt}}{\tan(\theta)^2 f_y} = 3.30 \text{ in}^2 \]

Minimum Area of Longitudinal Torsion Reinforcement (According to Eq.11-24 of ACI318),

\[ A_{l,min} = \frac{5 \times \sqrt{f_c \times A_{cp}} \times A'_{vt} \times P_h \times f_{yt}}{f_y} = 1.98 \text{ in}^2 \]

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.5
Provided Reinforcement, \( A_{sb} \) = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in^2

Number of Bars, \( n \) = 12

Provided Longitudinal Reinforcement, \( A_{l,prov} = A_{sb} \times n = 3.72 \text{ in}^2 \)

Check Validity = IF(\( A_{l,prov} \geq A_{l,req} \); "Valid"; "Invalid") = Valid
Chapter 1: Concrete Design
Precast Spandrel Beam for Combined Shear and Torsion

Calculation of Required Flexural Reinforcement

\[ R_n = \frac{M_u \times 12 \times 1000}{\phi_t b^2 d} = 530 \text{ psi} \]

\[ \rho = \frac{0.85 f'_c \left( \frac{1 - \sqrt{\frac{2R_n}{0.85 f'_c}}}{} \right)}{f_y} = 0.0095 \]

Area of Flexural Reinforcement, \( A_s = \rho b d = 6.92 \text{ in}^2 \)

Calculation of Total Bottom Reinforcement at Mid-Span

Percentage of Torsional Reinforcement Concentrated on Bottom Side, \( \text{Per} = 16 \% \)

Total Area of Bottom Reinforcement at Mid-Span,

\[ A_{sc\_Req} = A_{l\_Req} \times \text{Per}/100 + A_s = 7.45 \text{ in}^2 \]

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.11

Provided Reinforcement, \( A_{sb} = \text{TAB}("ACI/Bar"; Asb; Bar=Bar) = 1.56 \text{ in}^2 \)

Number of Bars, \( n = 5 \)

Total Area of Bottom Reinforcement, \( A_{sc\_Prov} = A_{sb} \times n = 7.80 \text{ in}^2 \)

Check Validity = IF(\( A_{sc\_Prov}=A_{sc\_Req} \); "Valid"; "Invalid") = Valid

Design Summary

Total Required Area for Torsion & Shear per Stirrups Spacing,

\[ A'_v = A'_v = 0.037 \text{ in}^2 \text{ per in per leg} \]

Provided Stirrups Spacing, \( s\_Prov = s\_Prov = 5.00 \text{ in} \)

Provided Longitudinal Reinforcement, \( A_{l\_Prov} = A_{l\_Prov} = 3.72 \text{ in}^2 \)

Total Area of Bottom Reinforcement, \( A_{sc\_Prov} = A_{sc\_Prov} = 7.80 \text{ in}^2 \)
Design of Beam Ledge as per ACI 318-11 Chapters 9 & 11

System

Width of Beam, \( b = 7.0 \text{ in} \)
Height of Beam, \( h = 36.0 \text{ in} \)
Width of Beam Ledge, \( b_L = 6.0 \text{ in} \)
Height of Beam Ledge, \( h_L = 12.0 \text{ in} \)
Concrete Cover, \( c_0 = 1.25 \text{ in} \)
Width of Bearing Pad, \( W = 4.5 \text{ in} \)
Length of Bearing Pad, \( L = 4.5 \text{ in} \)
Thickness of Bearing Pad, \( t_b = 0.3 \text{ in} \)
Gap Spacing, \( a_s = 1.0 \text{ in} \)
Shear Spacing, \( a_v = \frac{2}{3} \times L + a_s = 4.0 \text{ in} \)
Flexural Spacing, \( a_f = a_v + c_0 = 5.25 \text{ in} \)
Effective Width According to Shear Requirements, \( b_{ws} = W + 4 \times a_v = 20.5 \text{ in} \)
Effective Width According to Flexural Requirements, \( b_{wf} = W + 5 \times a_f = 30.8 \text{ in} \)
Effective Depth of Beam Ledge, \( d_L = h_L - c_0 = 10.75 \text{ in} \)

Load

Dead Load, \( P_D = 11.0 \text{ kips} \)
Live Load, \( P_L = 6.5 \text{ kips} \)
Service Load, \( P = P_D + P_L = 17.5 \text{ kips} \)
Ultimate Load, \( P_u = 1.2 \times P_D + 1.6 \times P_L = 23.6 \text{ kips} \)
Material Properties
Concrete Strength, $f'_c$ = 5000 psi
Yield Strength of Reinforcement, $f_y$ = 60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\phi_s$ = 0.75
Bearing Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\phi_b$ = 0.65
Modification Factor for Lightweight Concrete, $\lambda$ = 1.00
Friction Factor (According to Cl.11.6.4.3 of ACI318), $\mu$ = 1.4 * $\lambda$ = 1.40
Maximum Service Load for Bearing Pads, $q$ = 1000 psi

Check Bearing Plate Dimension
Capacity of Bearing Plate, $B_p$ = $W * L * q /1000$ = 20.25 kips
Check Validity = IF( $B_p > P$; "Valid" ; "Increase Dimension" ) = Valid

Check Concrete Bearing Strength
Bearing Strength of Concrete, $\phi P_{nb}$ = $\phi_b * 0.85 * f'_c * L * W /1000$ = 55.9 kips
Check Validity = IF( $\phi P_{nb} > P_u$; "Valid" ; "Invalid" ) = Valid

Check Maximum Nominal Shear-Transfer by Effective Section
Nominal Shear by Effective Section (According to Cl.11.9.3.2.1 of ACI318),
$V_{n1}$ = 0.2 * $f'_c$ * $b_{ws} * d_L /1000$ = 220.4 kips
$V_{n2}$ = (480 + 0.08 * $f'_c$) * $b_{ws} * d_L /1000$ = 193.9 kips
$V_{n3}$ = 1600 * $b_{ws} * d_L /1000$ = 352.6 kips
$\phi V_n$ = $\phi_s * MIN(V_{n1}; V_{n2}; V_{n3})$ = 145.4 kips
Check Validity = IF( $\phi V_n > P_u$; "Valid" ; "Increase Dimension" ) = Valid

Determine Shear Friction Reinforcement ($A_{vf}$)
Required Reinforcement for Shear Friction (According to Cl.11.6.4.1 of ACI318),
$A_{vf}$ = $P_u * 1000 / (\phi_s * f_y * \mu)$ = 0.37 in² per bwf

Determine Direct Tension Reinforcement ($A_n$)
Required Reinforcement for Direct Tension (According to Cl.11.8.3.4 of ACI318),
$A_n$ = 0.2 * $P_u * 1000 / (\phi_s * f_y)$ = 0.10 in² per bwf

Determine Flexural Reinforcement ($A_f$)
$M_u$ = $P_u * a_f + 0.2 * P_u * (h_L - d_L)$ = 129.8 kip*in
Required Reinforcement for Flexural (According to Cl.11.8.3.3 of ACI318),
$A_f$ = $M_u * 1000 / (\phi_s * f_y * 0.8 * d_L)$ = 0.34 in² per bwf
Determine Primary Tension Reinforcement ($A_{sc}$)

Required Area of Reinforcement for Primary Tension (According to Cl.11.8.3.5 of ACI318),

$$A_{sc} = \text{MAX} \left( \frac{2}{3} \frac{A_{vf}}{b_{ws}} + \frac{A_{n}}{b_{wf}} \ ; \frac{A_{f}}{b_{wf}} + \frac{A_{n}}{b_{wf}} \right) = 0.015 \text{ in}^2 \text{ per in}$$

Minimum Area of Reinforcement for Primary Tension (According to Cl.11.8.5 of ACI318),

$$A_{sc_{min}} = 0.04 \frac{f'_c}{f_y} d_L = 0.036 \text{ in}^2 \text{ per in}$$

$$A_{sc_{req}} = \text{MAX} (A_{sc}; A_{sc_{min}}) = 0.036 \text{ in}^2 \text{ per in}$$

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.5

Spacing between Bars, s = 8.0 in

Provided Reinforcement, $A_{sb}$ = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in$^2$

Check Validity = IF($A_{sb}$/$A_{sc_{req}}$ > s; "Valid"; "Invalid") = Valid

Determine Horizontal Reinforcement ($A_{h}$)

Required Area of Reinforcement for Horizontal Shear (According to Cl.11.8.4 of ACI318),

$$A_{h} = 0.5 (A_{sc_{req}} - A_{n}/b_{wf}) = 0.016 \text{ in}^2 \text{ per in}$$

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.4

Provided Reinforcement, $A_{sb}$ = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in$^2$

Check Validity = IF($A_{sb}$/A_{h} > s; "Valid"; "Invalid") = Valid

Design Summary

Primary Tension Reinforcement, $A_{sc_{req}} = A_{sc_{req}} = 0.036 \text{ in}^2 \text{ per in}$

Horizontal Shear Reinforcement, $A_{h} = A_{h} = 0.016 \text{ in}^2 \text{ per in}$

Distribute in two-thirds of Effective Ledge Depth adjacent to $A_{sc}$
Design of Rectangular Section with Tension Reinforcement only as per ACI 318-11 Chapters 9 & 10

System
Width of Concrete Section, b = 12.0 in
Depth of Concrete Section, h = 16.0 in
Concrete Cover, co = 2.5 in
Effective Depth of Concrete Section, d = h - co = 16.0 - 2.5 = 13.5 in

Load
Bending Moment due to Dead Load, \( M_D = 56.0 \text{ kip*ft} \)
Bending Moment due to Live Load, \( M_L = 35.0 \text{ kip*ft} \)
Ultimate Bending Moment, \( M_U = (1.2 \cdot M_D) + (1.6 \cdot M_L) \) = 123.2 kip*ft

Material Properties
Concrete Strength, \( f'_{c} \) = 4000 psi
Yield Strength of Reinforcement, \( f_{y} \) = 60000 psi
Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi \) = 0.90
Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3), \( \beta_1 \) = IF\( (f'_{c}\leq4000;0.85;IF(f'_{c}\geq8000;0.65;1.05-0.00005\cdot f'_{c})) \) = 0.85

Area of Reinforcement
\( R_n = \frac{M_U \cdot 12000}{\Phi \cdot b \cdot d^2} \) = 751.1 psi
\( \rho = \frac{0.85 \cdot f'_{c}}{f_{y}} \left( 1 - \sqrt{1 - \frac{2 \cdot R_n}{0.85 \cdot f'_{c}}} \right) \) = 0.0143
Area of Reinforcement, \( A_s = \rho \cdot b \cdot d \) = 2.32 in²
Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

\[ A_{s_{\text{min}1}} = \frac{3 \sqrt{f'_c \cdot b \cdot d}}{f_y} = 0.51 \text{ in}^2 \]

\[ A_{s_{\text{min}2}} = \frac{200 \cdot b \cdot d}{f_y} = 0.54 \text{ in}^2 \]

\[ A_{s_{\text{min}}} = \text{MAX}(A_{s_{\text{min}1}}; A_{s_{\text{min}2}}) = 0.54 \text{ in}^2 \]

Required Area of Reinforcement, \( A_{sc\_Req} = \text{MAX}(A_s; A_{s_{\text{min}}}) = 2.32 \text{ in}^2 \)

Provided Reinforcement, Bar= \( \text{SEL}("ACI/Bar"; \text{Bar}; ) = \text{No.10} \)

Provided Reinforcement, \( A_{sb} = \text{TAB}("ACI/Bar"; \text{Asb}; \text{Bar}=\text{Bar}) = 1.27 \text{ in}^2 \)

Number of Bars, \( n = 2 \)

Vertical Reinforcement, \( A_{sc\_Prov} = A_{sb} \cdot n = 2.54 \text{ in}^2 \)

Check Validity= \( \text{IF}(A_{sc\_Prov} \geq A_{sc\_Req}; \"Valid\"; \"Invalid\") = \text{Valid} \)

Check Tension Controlled

Depth of Rectangular Stress Block, \( a = \frac{A_{sc\_Prov} \cdot f_y}{0.85 \cdot f'_c \cdot b} = 3.74 \text{ in} \)

Distance from Extreme Compression Fiber to Neutral Axis, \( c = a/\beta_1 = 4.40 \text{ in} \)

\( c/d = \frac{4.40}{13.5} = 0.326 \)

\( \text{IF}(c/d>0.375; \"Add Com. RFT\"; \"Tension Controlled\") = \text{Tension Controlled} \)

Design Summary

Required Area of Reinforcement, \( A_{sc} = A_{sc\_Prov} = 2.54 \text{ in}^2 \)
Design of Rectangular Section with Compression Reinforcement as per ACI 318-11 Chapters 9 & 10

System
- Width of Concrete Section, \( b = 12.0 \text{ in} \)
- Depth of Concrete Section, \( h = 32.5 \text{ in} \)
- Concrete Cover, \( c_o = 2.5 \text{ in} \)
- Effective Depth of Concrete Section to Extreme Layer, \( d_t = h - c_o = 32.5 - 2.5 = 30.0 \text{ in} \)
- Distance between C.G of Tension Reinforcement and Extreme Layer, \( s = 1.2 \text{ in} \)
- Effective Depth of Concrete Section to C.G of Tension Reinforcement, \( d = d_t - s = 28.8 \text{ in} \)
- Depth of Compression Reinforcement, \( d' = 2.5 \text{ in} \)

Load
- Bending Moment due to Dead Load, \( M_D = 430.0 \text{ kip*ft} \)
- Bending Moment due to Live Load, \( M_L = 175.0 \text{ kip*ft} \)
- Ultimate Bending Moment, \( M_U = (1.2 * M_D) + (1.6 * M_L) = 796.0 \text{ kip*ft} \)

Material Properties
- Concrete Strength, \( f'_c = 4000 \text{ psi} \)
- Yield Strength of Reinforcement, \( f_y = 60000 \text{ psi} \)
- Modulus of Elasticity of Reinforcement, \( E_s = 29000000 \text{ psi} \)
- Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = 0.90 \)
- Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3), \( \beta_1 = \text{IF}(f'_c \leq 4000; 0.85; \text{IF}(f'_c \geq 8000; 0.65; 1.05 - 0.00005* f'_c)) = 0.85 \)
Check If Compression Reinforcement is Required

\[ \omega = 0.31875 \beta_1 = 0.271 \]

\[ R_{nt} = \omega (1-0.59*\omega) f'_c = 910.7 \text{ pci} \]

\[ R_n = \frac{M_{IU} * 12000}{\phi * b * d_t^2} = 982.7 \text{ psi} \]

Compression Reinforcement is: \( \text{IF}(R_n > R_{nt}; \text{"Required"}; \text{"Not Required"}) = \text{Required} \)

Determine Required Moment Resisted by Compression Reinforcement

\[ \omega = 0.31875 \beta_1 = 0.271 \]

\[ \rho_t = 0.31875 * f'_c * \beta_1 / f_y = 0.01806 \]

\[ \rho = \rho_t d_t / d = 0.01881 \]

\[ \omega = \rho f_y / f'_c = 0.28215 \]

Moment Resisted by Tension RFT, \( M_{nt} = \omega * (1-0.59*\omega) * \frac{f'_c * b * d^2}{12000} \)

\[ = 780.3 \text{ kip*ft} \]

Moment Resisted by Compression RFT, \( M'_n = M_{IU} / \Phi - M_{nt} = 104.1 \text{ kip*ft} \)

Required Area of Compression Reinforcement

\[ d'/c_{limit} = 1 - \frac{f_y}{E_s * 0.003} = 0.31 \]

\[ c_{limit} = \left( 1 - \frac{f_y}{E_s * 0.003} \right) d_t = 9.3 \text{ in} \]

\[ c_{cal} = 0.375 d_t = 11.3 \text{ in} \]

\[ d'/c_{cal} = d'/(c_{cal}) = 0.22 \]

\[ f'_{si} = \text{MIN}(0.003 * E_s * (1 - d'/c_{cal}); f_y) = 60000 \text{ psi} \]

\[ f'_s = \text{IF}(d'/c_{cal} \leq d'/c_{limit}; f_y; f'_{si}) = 60000 \text{ psi} \]

Required Reinforcement Area for Compression, \( A'_s = \frac{M'_n * 12000}{f'_s * (d - d')} = 0.79 \text{ in}^2 \)

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.6

Provided Reinforcement, \( A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar) = 0.44 \text{ in}^2 \)

Number of Bars, n= 2

Vertical Reinforcement, \( A'_{s, Prov} = A_{sb} * n = 0.88 \text{ in}^2 \)

Check Validity= \( \text{IF}(A'_{s, Prov} \geq A'_s; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \)

Required Reinforcement Area for Tension, \( A_s = A'_s + (\rho * b * d) = 7.29 \text{ in}^2 \)
Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

\[ A_{s\_min1} = \frac{3 \times \sqrt{f'_c} \times b \times d}{f_y} = 1.09 \text{ in}^2 \]

\[ A_{s\_min2} = \frac{200 \times b \times d}{f_y} = 1.15 \text{ in}^2 \]

\[ A_{s\_min} = \text{MAX}(A_{s\_min1} ; A_{s\_min2}) = 1.15 \text{ in}^2 \]

Required Area of Reinforcement, \( A_{sc\_Req} = \text{MAX}(A_s ; A_{s\_min}) = 7.29 \text{ in}^2 \)

Provided Reinforcement, Bar= \( \text{SEL("ACI/Bar"; Bar; )} = \text{No.10} \)

Provided Reinforcement, \( A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar}) = 1.27 \text{ in}^2 \)

Number of Bars, \( n = 6 \)

Vertical Reinforcement, \( A_{sc\_Prov} = A_{sb} \times n = 7.62 \text{ in}^2 \)

Check Validity= \( \text{IF}(A_{sc\_Prov} \geq A_{sc\_Req}; "Valid"; "Invalid") = \text{Valid} \)

Design Summary

Required Reinforcement Area for Compression, \( A'_s = A'_{s\_Prov} = 0.88 \text{ in}^2 \)

Required Area of Reinforcement, \( A_{sc} = A_{sc\_Prov} = 7.62 \text{ in}^2 \)
Design of Shear Reinforcement for Section Subject to Shear & Axial Compression As per ACI318-11

System
Width of Concrete Section, \( b = 12.0 \) in
Depth of Concrete Section, \( h = 16.0 \) in
Concrete Cover, \( c_o = 2.25 \) in
Effective Depth of Concrete Section, \( d = h - c_o = 13.75 \) in

Load
Shear Force due to Dead Load, \( V_D = 10.0 \) kips
Shear Force due to Live Load, \( V_L = 5.0 \) kips
Ultimate Shear Force, \( V_u = (1.2 \times V_D) + (1.6 \times V_L) = 20.0 \) kips
Axial Compression Force due to Dead Load, \( N_D = 4.2 \) kips
Axial Compression Force due to Live Load, \( N_L = 3.1 \) kips
Ultimate Axial Compression Force, \( N_u = (1.2 \times N_D) + (1.6 \times N_L) = 10.0 \) kips

Material Properties
Concrete Strength, \( f'_c = 4000 \) psi
Yield Strength of Reinforcement, \( f_y = 60000 \) psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \phi = 0.75 \)
Modification Factor for Lightweight Concrete, \( \lambda = 1.00 \)
Determine Concrete Shear Strength

Nominal Shear Strength provided by Concrete (According to Eq. 11-4 of ACI 318),

\[ V_c = 2 \left( 1 + \frac{N_u \cdot 1000}{2000 \cdot h \cdot b} \right) \cdot \lambda \cdot \sqrt{f_{cc} \cdot b \cdot d} \frac{1000}{1000} = 21.4 \text{ kips} \]

Shear Reinforcement is: \( \text{IF}(V_u > \Phi \cdot V_c; \text{"Required";} \text{"Not Required"}) = \text{Required} \)

Determine Area of Shear Reinforcement

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI 318),

\[ V_s = \frac{V_u - \Phi \cdot V_c}{\Phi} = 5.3 \text{ kips} \]

Maximum Allowable Shear Strength provided by Reinforcement (According to Cl.11.4.7.9 of ACI 318),

\[ V_{s_{\text{max}}} = 8 \cdot \lambda \cdot \sqrt{f_{cc} \cdot b \cdot d} \frac{1000}{1000} = 83.5 \text{ kips} \]

IF\( (V_s > V_{s_{\text{max}}}; \text{"Increase Beam Dimension";} \text{"OK";}) = \text{OK} \)

Spacing of Provided Stirrups, \( s = 6.75 \text{ in} \)

Required Area of Reinforcement, \( A_v = \frac{V_s \cdot s \cdot 1000}{f_y \cdot d} = 0.04 \text{ in}^2 \)

Minimum Area of Reinforcement (According to Cl.11.4.6.3 of ACI 318),

\[ A_{v_{\text{min1}}} = \frac{0.75 \cdot \sqrt{f_{cc} \cdot b \cdot s}}{f_y} = 0.06 \text{ in}^2 \]

\[ A_{v_{\text{min2}}} = \frac{50 \cdot b \cdot s}{f_y} = 0.07 \text{ in}^2 \]

\[ A_{v_{\text{min}}} = \text{MAX}(A_{v_{\text{min1}}}; A_{v_{\text{min2}}}) = 0.07 \text{ in}^2 \]

Required Area of Reinforcement, \( A_{v_{\text{vc Req}}} = \text{MAX}(A_v; A_{v_{\text{min}}}) = 0.07 \text{ in}^2 \)

Provided Reinforcement, Bar = \( \text{SEL("ACI/Bar"; Bar; \() = \text{No.3} \)

Provided Reinforcement, \( A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar)} = 0.11 \text{ in}^2 \)

Number of Stirrups, \( n = 1 \)

Provided Area of Reinforcement, \( A_{v_{\text{vc Prov}}} = A_{sb} \cdot n \cdot 2 = 0.22 \text{ in}^2 \)

Check Validity = \( \text{IF}(A_{v_{\text{vc Prov}}} \geq A_{v_{\text{vc Req}}}; \text{"Valid";} \text{"Invalid"}) = \text{Valid} \)
Determine Maximum Permissible Spacing of Stirrups

Allowable Shear Strength provided by Reinforcement for Spacing Limit (According to Cl.11.4.5.3 of ACI318),

\[ V_{s\_limit} = 4 \lambda \sqrt{f'_c b d} / 1000 \]

= 41.7 kips

Factor for Maximum Spacing of Stirrups, Fac = IF\(V_s \leq V_{s\_limit}\) ; 1; 0.5 = 1.0

Maximum Spacing of Stirrups (According to Cl.11.4.5.1 of ACI318),

\[ s_{\max} = \text{MIN}(d/2;24) \times \text{Fac} \]

= 6.88 in

Check Validity = IF\(s \leq s_{\max}\) ; "Valid"; "Invalid" = Valid

Design Summary

Provided Area of Shear Reinforcement, \(A_{vc\_Prov}\) = \(A_{vc\_Prov}\) = 0.22 in

Spacing of Stirrups, s = s = 6.75 in
Design of Shear Reinforcement for Section Subject to Shear & Axial Tension As per ACI318-11

System
- Width of Concrete Section, \( b = 10.5 \text{ in} \)
- Depth of Concrete Section, \( h = 18.0 \text{ in} \)
- Concrete Cover, \( c_o = 2.0 \text{ in} \)
- Effective Depth of Concrete Section, \( d = h - c_o = 16.0 \text{ in} \)

Load
- Shear Force due to Dead Load, \( V_D = 12.8 \text{ kips} \)
- Shear Force due to Live Load, \( V_L = 9.0 \text{ kips} \)
- Ultimate Shear Force, \( V_u = (1.2 \times V_D) + (1.6 \times V_L) = 29.8 \text{ kips} \)
- Axial Tension Force due to Dead Load, \( N_D = -2.0 \text{ kips} \)
- Axial Tension Force due to Live Load, \( N_L = -15.2 \text{ kips} \)
- Ultimate Axial Tension Force, \( N_u = (1.2 \times N_D) + (1.6 \times N_L) = -26.7 \text{ kips} \)

Material Properties
- Concrete Strength, \( f'_c = 3600 \text{ psi} \)
- Yield Strength of Reinforcement, \( f_y = 40000 \text{ psi} \)
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = 0.75 \)
- Modification Factor for Lightweight Concrete, \( \lambda = 0.85 \)

Determine Concrete Shear Strength
Nominal Shear Strength provided by Concrete (According to Eq. 11-8 of ACI318),
\[
V_c = 2 \times \left( \frac{N_u \times 1000}{500 \times h \times b} \right) \times \lambda \times \sqrt{\frac{f'_c \times b \times d}{1000}} = 12.3 \text{ kips}
\]
Shear Reinforcement is: \( \text{IF}(V_u > \Phi \times V_c; "Required", "Not Required") = Required \)
Determine Area of Shear Reinforcement

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),
\[ V_s = \frac{V_u - \phi \cdot V_c}{\phi} = \text{27.4 kips} \]

Maximum Allowable Shear Strength provided by Reinforcement (According to Cl.11.4.7.9 of ACI318),
\[ V_{s,\text{max}} = 8 \cdot \lambda \cdot \frac{\sqrt{f_c} \cdot b \cdot d}{1000} = \text{68.5 kips} \]

IF\( V_s > V_{s,\text{max}} \) ; "Increase Beam Dimension"; "OK"

Spacing of Provided Stirrups, \( s = \text{5.0 in} \)

Required Area of Reinforcement, \( A_v = \frac{V_s \cdot s \cdot 1000}{f_y \cdot d} = \text{0.21 in}^2 \)

Minimum Area of Reinforcement (According to Cl.11.4.6.3 of ACI318),
\[ A_{v,\text{min1}} = \frac{0.75 \cdot \sqrt{f_c} \cdot b \cdot s}{f_y} = \text{0.06 in}^2 \]
\[ A_{v,\text{min2}} = \frac{50 \cdot b \cdot s}{f_y} = \text{0.07 in}^2 \]

\[ A_{v,\text{min}} = \text{MAX}(A_{v,\text{min1}}; A_{v,\text{min2}}) = \text{0.07 in}^2 \]

Required Area of Reinforcement, \( A_{v,\text{req}} = \text{MAX}(A_v; A_{v,\text{min}}) = \text{0.21 in}^2 \)

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.3

Provided Reinforcement, \( A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 in}^2 \)

Number of Stirrups, n= 1

Provided Area of Reinforcement, \( A_{v,\text{prov}} = A_{sb} \cdot n \cdot 2 = \text{0.22 in}^2 \)

Check Validity= IF\( A_{v,\text{prov}} \geq A_{v,\text{req}} \); "Valid"; "Invalid"

Determine Maximum Permissible Spacing of Stirrups

Allowable Shear Strength provided by Reinforcement for Spacing Limit (According to Cl.11.4.5.3 of ACI318),
\[ V_{s,\text{limit}} = 4 \cdot \lambda \cdot \frac{\sqrt{f_c} \cdot b \cdot d}{1000} = \text{34.3 kips} \]

Factor for Maximum Spacing of Stirrups, \( F_{\text{ac}} = \text{IF}(V_s \leq V_{s,\text{limit}}; 1; 0.5) = \text{1.0} \)

Maximum Spacing of Stirrups (According to Cl.11.4.5.1 of ACI318),
\[ s_{\text{max}} = \text{MIN}(d / 2; 24) \cdot F_{\text{ac}} = \text{8.00 in} \]

Check Validity= IF\( s \leq s_{\text{max}} \); "Valid"; "Invalid"

Design Summary

Provided Area of Shear Reinforcement, \( A_{v,\text{prov}} = A_{v,\text{prov}} = \text{0.22 in}^2 \)

Spacing of Stirrups, \( s = \text{5.00 in} \)
Calculation of Deflection of Shored Nonprestressed Simple Support Concrete Composite Section

As per ACI318-11 Chapter 9

System

Beam Span, L = 26.0 ft
Beam Spacing, S = 8.0 ft
Width of Precast Beam, b = 12.0 in
Depth of Precast Beam, h = 20.0 in
Thickness of Cast in Situ Slab, h_f = 4.0 in
Area of Tension Reinforcement for Precast Beam, A_s = 3.00 in²
Concrete Cover for Precast Beam, c_o = 2.5 in
Effective Width of Slab, b_e1 = L*12 / 4 = 78.0 in
Effective Width of Slab, b_e2 = S*12 = 96.0 in
Effective Width of Slab, b_e3 = 16*h_f + b = 76.0 in
Effective Width of Slab, b_e = MIN(b_e1; b_e2; b_e3) = 76.0 in

Material Properties

Concrete Strength of Cast in Situ Slab, f'_{c1} = 3000 psi
Concrete Strength of Precast Beam, f'_{c2} = 4000 psi
Yield Strength of Reinforcement, f_y = 40000 psi
Modulus of Elasticity of Reinforcement, E_s = 29000000 psi
Modification Factor for Lightweight Concrete, \lambda = 1.00
Concrete Density, w_c = 150 psi
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Deflection of Shored Composite Section

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Load

Superimposed Dead Load, SDL = 10.00 psf
Live Load, LL = 75.00 psf

Dead Load per Unit Length for Slab, \( w_{d1} = SDL \times \frac{S + w_c \times S \times 12 \times h_l}{144} \) = 480.0 lb/ft
Dead Load per Unit Length for Beam, \( w_{d2} = w_c \times b \times h / 144 \) = 250.0 lb/ft
Live Load per Unit Length, \( w_l = LL \times S \) = 600.0 lb/ft

Percentage of Sustained Live Load, \( S_u \) = 20%

Bending Moment of Dead Load 1, \( M_{D1} = \frac{1}{1000} \times w_{d1} \times L^2 / 8 \) = 40.6 kip*ft
Bending Moment of Dead Load 2, \( M_{D2} = \frac{1}{1000} \times w_{d2} \times L^2 / 8 \) = 21.1 kip*ft
Bending Moment of Live Load, \( M_L = \frac{1}{1000} \times w_l \times L^2 / 8 \) = 50.7 kip*ft
Bending Moment of Sustained Load, \( M_{sus} = M_{D1} + M_{D2} + (S_u / 100) \times M_L \) = 71.8 kip*ft

Calculation of Modular Ratio

For Cast in Situ Slab:

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI 318),
\( E_{c1} = w_c^{1.5} \times 33 \times \sqrt{f'_{c1}} \) = 3320561 psi

Modulus of rupture (According to Eq. 9-10 of ACI 318), \( f_{r1} = 7.5 \times \lambda \times \sqrt{f'_{c1}} \) = 411 psi

For Precast Beam:

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI 318),
\( E_{c2} = w_c^{1.5} \times 33 \times \sqrt{f'_{c2}} \) = 3834254 psi

Modulus of rupture (According to Eq. 9-10 of ACI 318), \( f_{r2} = 7.5 \times \lambda \times \sqrt{f'_{c2}} \) = 474 psi

\( n_c = \frac{E_{c2}}{E_{c1}} \) = 1.15
\( n_s = \frac{E_s}{E_{c2}} \) = 7.56

Width of Slab considering relative Concrete Strength, \( b_s = \frac{b e}{n_c} \) = 66.09 in

Calculation of Moment of Inertia for Cracked Section

For Precast Beam

Effective Depth of Section, \( d = \) 17.5 in

\( I_{g1} = \frac{b \times h^3}{12} \) = 8000 in^4

\( B = \frac{b}{(n_s \times A_s)} \) = 0.53 in

\( k_d = \frac{\sqrt{2 \times d \times B + 1} - 1}{B} \) = 6.5 in

\( I_{cr1} = \frac{b \times k_d^3}{3} + n_s \times A_s \times (d - k_d)^2 \) = 3842.8 in^4
For Composite Section

Effective Depth of Section, \( d = (h + hf) - co \) = 21.5 in

\( h1 = h + hf \) = 24.0 in

\( bs1 = bs - b \) = 54.1 in

Distance from Centroidal Axis of Gross Section to Tension Face,

\[
yt = \frac{1}{h1} \left( \frac{bs1 * hf}{2} + \frac{b * h1}{2} \right) = 16.3 \text{ in}
\]

\[
l_g2 = \frac{bs1 * hf^3}{12} + \frac{b * h1^3}{12} + bs1 * hf \left( h + \frac{hf}{2} - yt \right)^2 + b * h1 \left( yt - \frac{h1}{2} \right)^2 = 26468.49 \text{ in}^4
\]

\[
B = \frac{bs}{(n_s * As)} = 2.91 \text{ 1/in}
\]

\[
k_d = \frac{\sqrt{2 * d * B + 1} - 1}{B} = 3.5 \text{ in}
\]

\[
l_{cr2} = \frac{bs * kd^3}{3} + n_s * As * (d - kd)^2 = 8292.9 \text{ in}^4
\]

Ratio between Cracking & Gross Inertia, \( r = \left( \frac{l_{g1}}{l_{g2}} \right) / 2 \) = 0.383

Cracking Moment (According to Eq. 9-9 of ACI318),

\[
M_{cr1} = \frac{f_{cr} * l_{g1}}{h / 2 * 12000} = 31.60 \text{ kip*ft}
\]

\[
M_{cr2} = \frac{f_{cr} * l_{g2}}{yt * 12000} = 64.14 \text{ kip*ft}
\]

Effective Moment of Inertia for Composite Section,

\[
l_{e1,2} = \left( \frac{M_{cr1}}{M_{D1} + M_{D2}} \right)^3 * l_{g1} + \left( 1 - \left( \frac{M_{cr1}}{M_{D1} + M_{D2}} \right)^3 \right) * l_{cr1} = 4401 \text{ in}^4
\]

\[
l_{ed,1} = \left( \frac{M_{cr2}}{M_{D1} + M_{D2} + M_L} \right)^3 * l_{g2} + \left( 1 - \left( \frac{M_{cr2}}{M_{D1} + M_{D2} + M_L} \right)^3 \right) * l_{cr2} = 11670 \text{ in}^4
\]

Check Validity = IF(\( l_{e1,2} < l_{g1} \); "Valid"; "Invalid") = Valid

**Short Term Deflection**

Short Term Deflection of composite section Due to Dead Load,

\[
\Delta_{t1,2} = \frac{5 * (M_{D1} + M_{D2}) * L^2 * 12^3}{48 * E_{c2} * l_{g2} / 1000} = 0.074 \text{ in}
\]
Deflection Due to Shrinkage
For Simple Span, \( K_{sh} = 0.125 \)

\[ \rho = \frac{A_s \times 100}{b \times d} = 1.16\% \]

(According to Fig.10-3 of PCA Note on ACI318), \( A_{sh} = 0.789 \)

Time Dependant Shrinkage Strain, \( \varepsilon_{sht} = 400 \times 10^{-6} \)

Deflection Due to Shrinkage, \( \Delta_{sh} = 0.64 \times K_{sh} \times A_{sh} \times \varepsilon_{sht} \times L^2 \times 12^2 \times r / h \) = 0.047 in

Deflection Due to Creep
For No Compression Reinforcement Factor of, \( k_r = 0.85 \)

Average Creep Coefficient (According to Cl.2.3.4 of ACI435), \( C_u = 1.67 \)

Deflection Due to Creep, \( \Delta_{cp} = C_u \times \Delta_{i1,2} \times k_r \) = 0.105 in

Deflection Due to Live Load
Deflection Due to Live Load, \( \Delta_L = \frac{5 \times (M_{D1} + M_{D2} + M_L) \times L^2 \times 12^3}{48 \times E \times c^2 \times I_{ed,l} / 1000} - \Delta_{i1,2} \) = 0.232 in

Deflection Due to Creep Sustained Live Load, \( \Delta_{cp, L} = \frac{C_u \times S_{us}}{100} \times \Delta_L \times k_r \) = 0.066 in

Total Long Term Deflection
Total Deflection, \( \Delta_u = \Delta_{i1,2} \times 3.53 + \Delta_{sh} + \Delta_{cp} + \Delta_L \) = 0.65 in

Calculation Summary
Total Deflection, \( \Delta_u = \Delta_{i1,2} \times 3.53 + \Delta_{sh} + \Delta_{cp} + \Delta_L \) = 0.65 in
Design of Flanged Section with Tension Reinforcement only as per ACI 318-11 Chapters 9 & 10

System

- Width of Concrete Flange, \( b_f = 30.0 \) in
- Width of Concrete Web, \( b_w = 10.0 \) in
- Depth of Concrete Section, \( h = 20.0 \) in
- Thickness of Top Flange, \( h_f = 2.5 \) in
- Concrete Cover, \( c_o = 1.0 \) in
- Effective Depth of Concrete Section, \( d = h - c_o = 20.0 - 1.0 = 19.0 \) in

Load

- Bending Moment due to Dead Load, \( M_D = 72.0 \) kip*ft
- Bending Moment due to Live Load, \( M_L = 196.0 \) kip*ft
- Ultimate Bending Moment, \( M_U = (1.2 \times M_D) + (1.6 \times M_L) = 400.0 \) kip*ft

Material Properties

- Concrete Strength, \( f'_c = 4000 \) psi
- Yield Strength of Reinforcement, \( f_y = 60000 \) psi
- Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = 0.90 \)
- Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3 of ACI318), \( \beta_1 = \text{IF}(f'_c \leq 4000; 0.85; \text{IF}(f'_c \geq 8000; 0.65; 1.05 - 0.00005 \times f'_c)) = 0.85 \)
Design as Flanged Section

Compressive Strength of Flange, $C_f = 0.85 f'_c * h_f * \frac{b_f - b_w}{1000} = 170.0 \text{ kips}$

Area of Reinforcement for Flange in Compression, $A_{sf} = \frac{C_f * 1000}{f_y} = 2.83 \text{ in}^2$

Nominal Moment for Flange, $M_{nf} = \frac{A_{sf} * f_y * \left( h_f - \frac{d}{2} \right)}{12000} = 251.2 \text{ kip*ft}$

Nominal Moment for Web, $M_{nw} = M_U / \phi - M_{nf} = 193.24 \text{ kip*ft}$

$R_{nw} = \frac{M_{nw} * 12000}{\phi * b_w * d^2} = 713.7 \text{ psi}$

$\rho_w = 0.85 * \frac{f'_c}{f_y} * \left( 1 - \sqrt{1 - \frac{2 * R_{nw}}{0.85 * f'_c}} \right) = 0.0135$

Area of Reinforcement for Web in Compression, $A_{sw} = \rho_w * b_w * d = 2.56 \text{ in}^2$

Required Area of Reinforcement, $A_{s_R} = A_{sf} + A_{sw} = 5.39 \text{ in}^2$

Depth of Rectangular Stress Block for Web, $a_w = \frac{A_{sw} * f_y}{0.85 * f'_c * b_w} = 4.52 \text{ in}$

Design as Rectangular Section

$R_n = \frac{M_U * 12000}{\phi * b_f * d^2} = 492.46 \text{ psi}$

$\rho = 0.85 * \frac{f'_c}{f_y} * \left( 1 - \sqrt{1 - \frac{2 * R_n}{0.85 * f'_c}} \right) = 0.0089$

Area of Reinforcement, $A_{s_R} = \rho * b_f * d = 5.07 \text{ in}^2$

Depth of Rectangular Stress Block, $a = \frac{A_{s_R} * f_y}{0.85 * f'_c * b_f} = 2.98 \text{ in}$
Section Type and Reinforcement

Section Design as: \( \text{IF}(a > h; \ \text{"Flanged Sec."}; \ \text{"Rectangular Sec."}) \) = Flanged Sec.

Area of Reinforcement, \( A_s \) = \( \text{IF}(a > h; \ A_{s_T}; \ A_{s_R}) \) = 5.39 in\(^2\)

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

\[
A_{s_{\text{min}1}} = \frac{3 \sqrt{f'_c \cdot b_f \cdot d}}{f_y} = 1.80 \text{ in}^2
\]

\[
A_{s_{\text{min}2}} = \frac{200 \cdot b_f \cdot d}{f_y} = 1.90 \text{ in}^2
\]

\[
A_{s_{\text{min}}} = \text{MAX}(A_{s_{\text{min}1}}; \ A_{s_{\text{min}2}}) = 1.90 \text{ in}^2
\]

Required Area of Reinforcement, \( A_{s_{\text{Req}}} \) = MAX(\( A_s \); \( A_{s_{\text{min}}} \)) = 5.39 in\(^2\)

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.10

Provided Reinforcement, \( A_{sb} \) = TAB("ACI/Bar"; Asb; Bar=Bar) = 1.27 in\(^2\)

Number of Bars, \( n \) = 5

Vertical Reinforcement, \( A_{s_{\text{Prov}}} \) = \( A_{sb} \cdot n \) = 6.35 in\(^2\)

Check Validity = \( \text{IF}(A_{s_{\text{Prov}}}; A_{s_{\text{Req}}}; \ "Valid"; \ "Invalid") = \) Valid

Check Tension Controlled

Distance from Extreme Compression Fiber to Neutral Axis,

\[
c = \text{IF}(a > h; \ a_w / \beta_1; a / \beta_1) = 5.32 \text{ in}
\]

\[
c/d = c / d = 5.32 / 19.0 = 0.280
\]

IF(c/d>0.375; "Add Com. RFT"; "Tension Controlled") = Tension Controlled

Design Summary

Required Area of Reinforcement, \( A_{s_{\text{sc}}} \) = \( A_{s_{\text{scProv}}} \) = 6.35 in\(^2\)
Cracking Moment Strength for Prestressed Sections as per ACI 318-11 Chapter 18

System

- Width of Concrete Section, \( b = 12.0 \text{ in} \)
- Depth of Concrete Section, \( h = 24.0 \text{ in} \)
- Concrete Cover, \( c_0 = 2.0 \text{ in} \)
- Effective Depth of Concrete Section, \( d = h - c_0 = 24.0 - 2.0 = 22.0 \text{ in} \)
- Number of Strands, \( n = 6.0 \)
- Area of One Strand, \( A_s = 0.153 \text{ in}^2 \)

Material Properties

- Concrete Strength, \( f'_c = 5000 \text{ psi} \)
- Tensile Strength of Prestressed Steel, \( f_{pu} = 270000 \text{ psi} \)
- Jacking Stress, \( J_s = 0.7 * f_{pu} = 189000 \text{ psi} \)
- Percentage of Losses, \( L_s = 20.00 \% \)
- Modification Factor for Lightweight Concrete, \( \lambda = 1.00 \)
- Modulus of Rupture (According to Eq. 9-10 of ACI318), \( f_r = 530 \text{ psi} \)

Calculation of Cracking Moment Strength

\[
M_{cr} = \left( \frac{f_r}{1000} + \frac{P_{se}}{A_c} \right) * \frac{S_b}{12} + \frac{P_{se} * e}{12}
\]

Calculation Summary

- Cracking Moment Strength, \( M_{cr} = 212.8 \text{ kip*ft} \)
Flexural Strength of Prestressed Member Using Approximate Value of $f_{ps}$ As per ACI 318-11

**System**
- Width of Concrete Section, $b =$ 12.0 in
- Depth of Concrete Section, $h =$ 24.0 in
- Concrete Cover, $c_o =$ 2.0 in
- Effective Depth of Concrete Section, $d =$ $h - c_o = 24.0 - 2.0 = 22.0$ in
- Number of Strands, $n =$ 6
- Area of One Strand, $A_{s} =$ 0.153 in$^2$

**Material Properties**
- Concrete Strength, $f'_{c} =$ 5000 psi
- Tensile Strength of Prestressed Steel, $f_{pu} =$ 270000 psi
- Yield Strength of Prestressed Steel, $f_{py} = 0.9 * f_{pu} = 243000$ psi
- Factor for Type of Prestressing Steel (According to Cl.18.7.2 of ACI318), $\gamma_p =$ 0.28
- Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3 of ACI318), $\beta_1 =$ $IF(f'_c \leq 4000; 0.85; IF(f'_c > 8000; 0.65; 1.05 - 0.00005 * f'_c)) = 0.80$
Calculation of Stress for Prestressed Reinforcement

Prestressed Reinforcement Ratio, $\rho_p = \frac{n * A_s}{(b * d)} = 0.00348$

Prestressing Force (According to Eq. 18-1 of ACI 318),

$$f_{ps} = \frac{f_{pu}}{1000} * \left(1 - \frac{\gamma_p}{\beta_1} \rho_p \frac{f_{pu}}{f'_c}\right) = 252 \text{ ksi}$$

Calculation of Nominal Moment Strength

Distance of Compression Block, $a = \frac{n * A_s * f_{ps}}{0.85 * b * f'_c / 1000} = 4.54 \text{ in}$

Nominal Moment Strength, $M_n = \frac{n * A_s * f_{ps}}{12} * \left(d - \frac{a}{2}\right) = 380.4 \text{ kip*ft}$

Calculation Summary

Nominal Moment Strength, $M_n = M_n = 380.4 \text{ kip*ft}$
Tension Controlled Limit for Prestressed Flexural Member as per ACI 318-11 Chapters 10 & 18

### System
- Width of Concrete Double Tee Section, $b = 84.0$ in
- Width of Web of Concrete Double Tee Section, $b_w = 15.5$ in
- Depth of Concrete Double Tee Section, $h = 32.0$ in
- Thickness of Concrete Top Slab, $h_f = 2.0$ in
- Concrete Cover, $c_o = 2.0$ in
- Concrete Cover to CG of Prestressed Steel, $c_{o_p} = 4.5$ in
- Effective Depth of Concrete Section, $d = h - c_o = 30.0$ in
- Effective Depth of Concrete Section, $d_{p} = h - c_{o_p} = 27.5$ in
- Number of Strands, $n = 22.0$
- Area of One Strand, $A_s = 0.153$ in$^2$

### Material Properties
- Concrete Strength, $f'_{c} =$ 5000 psi
- Tensile Strength of Prestressed Steel, $f_{pu} =$ 270000 psi
- Yield Strength of Prestressed Steel, $f_{py} = 0.9 * f_{pu} =$ 243000 psi
- Factor for Type of Prestressing Steel (According to Cl.18.7.2 of ACI318), $\gamma_p =$ 0.28
- Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3 of ACI318),
  $\beta_{1} = IF(f'_c \leq 4000; 0.85; IF(f'_c \geq 8000; 0.65; 1.05 - 0.00005* f'_c)) =$ 0.80
Chapter 1: Concrete Design

Tension Controlled Limit for Prestressed Flexural Member

Calculation of Stress in Prestressed Reinforcement

$$\omega_{pu} = \frac{(n * A_s) * f_{pu}}{(b + d_p + f_c)} = 0.079$$

Prestressing Force (According to Eq.18-1 of ACI318),

$$f_{ps} = f_{pu} \left(1 - \frac{\gamma_p}{\beta_1} \omega_{pu}\right) = 262535 \text{ psi}$$

Area of Reinforcement for Compression in Flange,

$$A_{pf} = 0.85 * h_f * f_c * (b-b_w) / f_{pu} = 2.16 \text{ in}^2$$

Calculation of Depth of Concrete Stress Block

$$a_i = \frac{(n * A_s) * f_{ps}}{(0.85 * b * f_c)} = 2.48 \text{ in}$$

For $$a_i > h_f$$:

$$a_1 = \frac{(n * A_s - A_{pf}) * f_{ps}}{(0.85 * b_w * f_c)} = 4.81 \text{ in}$$

For $$a_i \leq h_f$$:

$$a_2 = a_i = 2.48 \text{ in}$$

$$a = \text{IF}(a_i > h_f; a_1; a_2) = 4.81 \text{ in}$$

$$c = a / \beta_1 = 6.01 \text{ in}$$

Check Tension Controlled

$$c/d = c / d = 0.200$$

IF($c/d > 0.375; "Compression Controlled"; "Tension Controlled") = Tension Controlled

Calculation Summary

Type of Section:

IF($c/d > 0.375; "Compression Controlled"; "Tension Controlled") = Tension Controlled
Estimating Prestress Losses as per ACI 318-11 Chapter 18

System

Area of Concrete Section, $A_c =$ 449 in$^2$
Depth of Concrete Section, $h =$ 24 in
Concrete Cover, $c_o =$ 2 in
Effective Depth of Concrete Section, $d =$ $h - c_o =$ 22 in
Moment of Inertia for Concrete Section, $I_c =$ 22469 in$^4$
Distance from Bottom Fiber to Neutral Axis, $y_b =$ 17.77 in
Distance from Top Fiber to Neutral Axis, $y_t =$ $h - y_b =$ 6.23 in
Number of Strands, $n =$ 8.0
Area of One Strand, $A_s =$ 0.153 in$^2$
Eccentricity of Strands, $e =$ 9.77 in
Volume per Surface Area, $V.S =$ 1.35 in
Average Relative Humidity, $RH =$ 75.00 %

Load

Factored Moment due to Dead Load, $M_D =$ 1617 kip*in
Factored Moment due to Superimposed Dead Load, $M_{SD} =$ 691 kip*in
Factored Moment due to Live Load, $M_L =$ 1382.00 kip*in
Chapter 1: Concrete Design

Prestress Losses

Material Properties

Concrete Strength, $f'_{ci} =$ 3500 psi
Concrete Strength, $f'_{c} =$ 5000 psi
Tensile Strength of Prestressed Steel, $f_{pu} =$ 270000 psi
Yield Strength of Prestressed Steel, $f_{py} =$ 0.9 * $f_{pu} = 243000 psi
Jacking Stress, $J_s =$ 0.74 * $f_{pu} = 199800 psi
Modification Factor for Lightweight Concrete, $\lambda =$ 1.00
Modulus of Rupture (According to Eq. 9-10 of ACI318), $f_r =$ 7.5 * $\lambda$ * $\sqrt{f'_{c}} = 530 psi
Concrete Density, $w_c =$ 150 psi

Calculation of Losses

1- Elastic Shortening of Concrete (ES)
Initial Force of Prestress, $P_{pi} =$ $J_s$ * ($n$ * $A_s$)/1000 = 244.6 kips
Prestress Type= SEL("ACI/Kes" ;Type; ) = Pretensioned

2- Creep of Concrete (CR)
Prestress Type= SEL("ACI/Kcr" ;Type; ) = Pretensioned
Factor of, $K_{cr} =$ TAB("ACI/Kcr" ;$K_{cr}$ ;Type=Type ) = 2.00
Creep Losses, CR=$K_{cr}$ * $E_s$ * $f_{cir}$ / $E_{ci}$ = 5.61 ksi

3- Shrinkage of Concrete (SH)
Prestress Type= SEL("ACI/Ksh" ;Type; ) = Pretensioned
Factor of, $K_{sh} =$ TAB("ACI/Ksh" ;$K_{sh}$ ;Type=Type ) = 1.00
Shrinkage Losses, SH=$8.2 * 10^{-6}$ * $K_{sh}$ * $E_s$ * (1 - 0.06 * V.S) * (100 - RH) = 5.37 ksi
4- Relaxation of Tendon (RE)

Prestress Type= SEL("ACI/KreJ" ;Type; ) = relaxation strand-Grade Low 270

Factor of, $K_{re}$ = TAB("ACI/KreJ";Kre ;Type=Type ) = 5000 psi

Factor of, J= TAB("ACI/KreJ";J ;Type=Type ) = 0.04

Ratio of $f_{pu}/f_{pu}$, $r$= SEL("ACI/r" ;r; ) = 0.74

Factor of, C= TAB("ACI/r" ;C ;r=r ) = 0.95

Relaxation of Tendon, RE= \[
\left( \frac{K_{re}}{1000}\right)^{-J}\left(\text{SH} + \text{CR} + \text{ES}\right) * C = 4.11 \text{ ksi}
\]

5- Total Allowance of Losses and Effective Prestress Force after all Losses

Total Allowance of Losses, $L_s$ = $ES + CR + SH + RE$ = 21 ksi

Effective Prestress Stress, $f_{se}$ = $J_s / 1000 - L_s$ = 179 ksi

Effective Prestress Force after All Losses, $P_e$ = $f_{se} * (n * A_s )$ = 219 kips

Calculation Summary

Total Allowance of Losses, $L_s$ = $L_s$ = 21 ksi

Effective Prestress Force after All Losses, $P_e$ = $P_e$ = 219 kips
Design Shear Reinforcement for Slab which to resist Punching Stress around Interior Square Column
As per ACI318-11 Chapter 11

**System**
- Column Dimension, c = 12.0 in
- Thickness of Concrete Slab, h = 7.5 in
- Concrete Cover, co = 1.5 in
- Effective Depth of Concrete Section, d = h - co = 7.5 - 1.5 = 6.0 in
- Bar Diameter of Shear Reinforcement, Dia = 0.375 in

**Load**
- Ultimate Shear Force, \( V_u \) = 120.0 kips

**Material Properties**
- Concrete Strength, \( f'_c \) = 4000 psi
- Yield Strength of Reinforcement, \( f_y \) = 60000 psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi \) = 0.75
- Modification Factor for Lightweight Concrete, \( \lambda \) = 1.00

**Determine Concrete Shear Strength**
- \( b_1 \) = c + d = 18.0 in
- Perimeter of Critical Section, \( b_0 \) = 4 * \( b_1 \) = 72.0 in
- Nominal Shear Strength provided by Concrete (According to Eq. 11-33 of ACI318), \( V_c \) =
  \[
  4 \times \lambda \times \sqrt{f'_c \times b_0 \times d / 1000} = 109.3 \text{ kips}
  \]
- Punching Shear Reinforcement is:
  \[
  \text{IF}(V_u > \Phi \times V_c; \text{"Required"}; \text{"Not Required"}) = \text{Required}
  \]
Determine Area of Shear Reinforcement

Minimum Effective Depth of Slab with Shear Reinforcement (According to Cl.11.11.3 of ACI318),
\[ d_{\text{min}} = \text{MIN}(6;16 \times \text{Dia}) \]

Effective Depth of Slab: IF\(d > d_{\text{min}}\); "Should Increase"; "OK"

Maximum Shear Strength of Slab with Shear Reinforcement (According to Cl.11.11.3.2 of ACI318),
\[ V_{n} = 6\sqrt{f'_{c}b_{0}d}/1000 \]

Validity : IF\(V_{u} > \Phi \times V_{n}\); "Not Valid"; "Valid"

Shear Strength provided by Concrete with Shear RFT (According to Cl.11.11.3.1 of ACI318),
\[ V_{ci} = 2\lambda \sqrt{f'_{c}b_{0}d}/1000 \]

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),
\[ V_{s} = \frac{V_{u} - \Phi \times V_{ci}}{\Phi} \]

Spacing of Provided Bars, \(s\) = 3.0 in

Required Area of Reinforcement, \(A_{v} = \frac{V_{s} \times s \times 1000}{f_{y} \times d} \)
\[ \text{} = 0.88 \text{ in}^{2} \]

Required Area of Reinforcement for each side of Column, \(A_{v_{\text{side}}} = A_{v}/4 \)
\[ \text{} = 0.22 \text{ in}^{2} \]

Perimeter of Critical Section where Shear Reinforcement may be terminated,
\[ b'_{0} = \frac{V_{u} \times 1000}{\Phi \times 2\lambda \sqrt{f'_{c}d}} \]
\[ \text{} = 210.8 \text{ in} \]

Distance from Column Face where Shear Reinforcement may be terminated,
\[ a = \left(\frac{b'_{0}}{4 - c}\right) / \sqrt{2} \]
\[ \text{} = 28.8 \text{ in} \]

Design Summary

Required Area of Reinforcement, \(A_{v} = A_{v} \)
\[ \text{} = 0.88 \text{ in}^{2} \]

Distance from Column Face where Shear Reinforcement may be terminated: \(a = 28.8 \text{ in} \)
Design of One Way Joist as per ACI 318-11 Chapters 9 & 11

System
- Width of Beam, b = 30.0 in
- Width of Joist, bj = 6.0 in
- Spacing between Joists, s = 36.0 in
- Slab Thickness, ts = 3.5 in
- Exterior Joist Span, Lne = 27.5 ft
- Interior Joist Span, Lni = 27.0 ft
- Concrete Cover, co = 1.25 in

Load
- Dead Load, DL = 130 psf
- Live Load, LL = 60 psf
- Ultimate Load, \( w_u = \frac{1.2 \times DL + 1.6 \times LL \times s}{1000 \times 12} \) = 0.756 kip/ft
Chapter 1: Concrete Design
One Way Joist

Material Properties

Concrete Strength, $f'_{c}$ = 4000 psi
Yield Strength of Reinforcement, $f_{y}$ = 60000 psi
Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi$ = 0.90
Modification Factor for Lightweight Concrete, $\lambda$ = 1.00
Factor for Rectangular Compressive Stress Block (According to Cl.10.2.7.3), $\beta_{1} = \text{IF}(f'_{c} \leq 4000; 0.85; \text{IF}(f'_{c} \geq 8000; 0.65; 1.05 - 0.00005*f'_{c})) = 0.85$

Moment Distribution for Joist

1. End Span

Edge Negative Moment for Exterior Joist, $M_{ne}$ = $\frac{w_{u} * L_{ne}^{2}}{24}$ = 23.8 kip*ft
Positive Moment for Exterior Joist, $M_{pe}$ = $\frac{w_{u} * L_{ne}^{2}}{14}$ = 40.8 kip*ft
Negative Moment for Exterior Joist, $M_{ne}$ = $\frac{w_{u} * (L_{ne} + L_{ni})^{2}}{10}$ = 56.1 kip*ft

2. Interior Spans

Negative Moment for Interior Joist, $M_{ni}$ = $\frac{w_{u} * L_{ni}^{2}}{11}$ = 50.1 kip*ft
Positive Moment for Interior Joist, $M_{pi}$ = $\frac{w_{u} * L_{ni}^{2}}{16}$ = 34.4 kip*ft

3. Maximum Moment

$M_{max}$ = MAX($M_{ne}$; $M_{pe}$; $M_{ne}$; $M_{ni}$; $M_{pi}$) = 56.1 kip*ft
Calculation of Required Depth for Joist

\[ \rho_t = 0.319 \cdot f_c \cdot \beta_1 / f_y = 0.01808 \]

For Reasonable Deflection Control, choose a Reinforcement Ratio (\(\rho\)) equal to about one-half (\(\rho_t\)).

Reinforcement Ratio, \(\rho\) = \(\rho_t / 2\) = 0.00904

\[ \omega = \rho_{*} f_y \]
\[ \omega = \rho_{*} f_y \]
\[ \omega = \rho_{*} f_y \]

Required Depth, \(d\) = \[ \sqrt{ \frac{M_{\text{max}} \cdot 12000}{\Phi \cdot b_j \cdot f_c \cdot \omega \cdot (1 - 0.59 \cdot \omega)}} \] = 15.8 in

Required Thickness, \(h_{\text{req}}\) = \(d + co\) = 17.1 in

\(h_{\text{min}}\) = \(\text{MAX}(L_{\text{ne}} \cdot 12 + b; L_{\text{mi}} \cdot 12 + b) / 18.5\) = 19.5 in

Provided Thickness, \(h\) = \(\text{MAX}(h_{\text{req}}; h_{\text{min}})\) = 19.5 in

Effective Depth of Joist, \(d_j\) = \(h - co\) = 18.25 in

Calculation of Required Reinforcement for Exterior Negative Moment of End Span (\(A_{sc1}\))

\[ R_{n1} = \frac{M_{\text{nee}} \cdot 12000}{\Phi \cdot b_j \cdot d_j^2} \] = 159 psi

Reinforcement Ratio, \(\rho_{1} = 0.85 \cdot \frac{f_c}{f_y} \left(1 - \sqrt{1 - 2 \cdot \frac{R_{n1}}{0.85 \cdot f_c}}\right) = 0.0027\)

Area of Reinforcement, \(A_{s1} = \rho_{1} \cdot b_j \cdot d_j = 0.30 \text{ in}^2\)

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

\[ A_{s_{\text{min}1}} = \frac{3 \cdot \sqrt{f_c \cdot b_j \cdot d_j}}{f_y} = 0.35 \text{ in}^2\]

\[ A_{s_{\text{min}2}} = \frac{200 \cdot b_j \cdot d_j}{f_y} = 0.36 \text{ in}^2\]

\[ A_{s_{\text{min}}} = \text{MAX}(A_{s_{\text{min}1}}; A_{s_{\text{min}2}}) = 0.36 \text{ in}^2\]

Required Area of Reinforcement, \(A_{sc1} = \text{MAX}(A_{s1}; A_{s_{\text{min}}}) = 0.36 \text{ in}^2\)

Provided Reinforcement, \(\text{Bar} = \text{SEL}(\text{"ACI/Bar"}; \text{Bar}; ) = \text{No.3}\)

Provided Reinforcement, \(A_{sb} = \text{TAB}(\text{"ACI/Bar"}; \text{Asb}; \text{Bar=Bar}) = 0.11 \text{ in}^2\)

Number of Bars, \(n = 4\)

Vertical Reinforcement, \(A_{sc1_{\text{Prov}}} = A_{sb} \cdot n = 0.44 \text{ in}^2\)

Check Validity = \(IF(A_{sc1_{\text{Prov}} \geq A_{sc1}}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid}\)
Calculation of Required Reinforcement for Positive Moment of End Span ($A_{sc2}$)

$$R_{n2} = \frac{M_{pe} \times 12000}{\Phi \times b_j \times d_j} = 272 \text{ psi}$$

Reinforcement Ratio, $\rho_2 = 0.85 \frac{f_p}{f_y} \left( 1 - \sqrt{1 - 2 \times \frac{R_{n2}}{0.85 \times f_p}} \right) = 0.0047$

Area of Reinforcement, $A_{s2} = \rho_2 \times b_j \times d_j = 0.51 \text{ in}^2$

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$$A_{s_{min1}} = \frac{3 \times \sqrt{f_p} \times b_j \times d_j}{f_y} = 0.35 \text{ in}^2$$

$$A_{s_{min2}} = \frac{200 \times b_j \times d_j}{f_y} = 0.36 \text{ in}^2$$

$$A_{s_{min}} = \text{MAX}(A_{s_{min1}}; A_{s_{min2}}) = 0.36 \text{ in}^2$$

Required Area of Reinforcement, $A_{sc2} = \text{MAX}(A_{s2}; A_{s_{min}}) = 0.51 \text{ in}^2$

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.5

Provided Reinforcement, $A_{sb} = \text{TAB}("ACI/Bar"; Asb; Bar=Bar) = 0.31 \text{ in}^2$

Number of Bars, $n = 2$

Vertical Reinforcement, $A_{sc2 Prov} = A_{sb} \times n = 0.62 \text{ in}^2$

Check Validity = IF($A_{sc2 Prov} \geq A_{sc2}$; "Valid"; "Invalid") = Valid

Calculation of Required Reinforcement for Interior Negative Moment of End Span ($A_{sc3}$)

$$R_{n3} = \frac{M_{ne} \times 12000}{\Phi \times b_j \times d_j} = 374 \text{ psi}$$

Reinforcement Ratio, $\rho_3 = 0.85 \frac{f_p}{f_y} \left( 1 - \sqrt{1 - 2 \times \frac{R_{n3}}{0.85 \times f_p}} \right) = 0.0066$

Area of Reinforcement, $A_{s3} = \rho_3 \times b_j \times d_j = 0.72 \text{ in}^2$

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$$A_{s_{min1}} = \frac{3 \times \sqrt{f_p} \times b_j \times d_j}{f_y} = 0.35 \text{ in}^2$$

$$A_{s_{min2}} = \frac{200 \times b_j \times d_j}{f_y} = 0.36 \text{ in}^2$$

$$A_{s_{min}} = \text{MAX}(A_{s_{min1}}; A_{s_{min2}}) = 0.36 \text{ in}^2$$

Required Area of Reinforcement, $A_{sc3} = \text{MAX}(A_{s3}; A_{s_{min}}) = 0.72 \text{ in}^2$
Chapter 1: Concrete Design
One Way Joist

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.5
Provided Reinforcement, $A_{sb}$ = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in$^2$
Number of Bars, n = 3
Vertical Reinforcement, $A_{sc3_{Prov}}$ = $A_{sb} \times n$ = 0.93 in$^2$
Check Validity = IF($A_{sc3_{Prov}} \geq A_{sc3}$; "Valid"; "Invalid") = Valid

Calculation of Required Reinforcement for Interior Negative Moment of Interior Span ($A_{sc4}$)

$$R_{n4} = \frac{M_{ni} \times 12000}{\phi \times b_j \times d_j^2} = 334 \text{ psi}$$

Reinforcement Ratio, $\rho_4$ =

$$0.85 \times \frac{f_c}{f_y} \left(1 - \sqrt{1 - 2 \times \frac{R_{n4}}{0.85 \times f_c}}\right) = 0.0059$$

Area of Reinforcement, $A_{s4}$ = $\rho_4 \times b_j \times d_j$ = 0.65 in$^2$

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$$A_{s\_min1} = \frac{3 \times \sqrt{\frac{f_c}{f_y} \times b_j \times d_j}}{200 \times b_j \times d_j} = 0.35 \text{ in}^2$$

$$A_{s\_min2} = \frac{200 \times b_j \times d_j}{f_y} = 0.36 \text{ in}^2$$

$$A_{s\_min} = \max(A_{s\_min1}; A_{s\_min2}) = 0.36 \text{ in}^2$$

Required Area of Reinforcement, $A_{sc4}$ = $\max(A_{s4}; A_{s\_min})$ = 0.65 in$^2$

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.5
Provided Reinforcement, $A_{sb}$ = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in$^2$
Number of Bars, n = 3
Vertical Reinforcement, $A_{sc4_{Prov}}$ = $A_{sb} \times n$ = 0.93 in$^2$
Check Validity = IF($A_{sc4_{Prov}} \geq A_{sc4}$; "Valid"; "Invalid") = Valid

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Calculation of Required Reinforcement for Interior Positive Moment of Interior Span ($A_{sc5}$)

$$R_n5 = \frac{M_{pl} \times 12000}{\phi \times b_j \times d_j} = 230 \text{ psi}$$

Reinforcement Ratio, $\rho_5 = 0.85 \times \frac{f'_c}{f_y} \left(1 - \sqrt{1 - 2 \times \frac{R_n5}{0.85 \times f'_c}}\right) = 0.0040$

Area of Reinforcement, $A_{s5} = \rho_5 \times b_j \times d_j = 0.44 \text{ in}^2$

Minimum Area of Reinforcement (According to Cl.10.5 of ACI318),

$$A_{s\_min1} = \frac{3 \times \sqrt{f'_c \times b_j \times d_j}}{f_y} = 0.35 \text{ in}^2$$

$$A_{s\_min2} = \frac{200 \times b_j \times d_j}{f_y} = 0.36 \text{ in}^2$$

$$A_{s\_min} = \text{MAX}(A_{s\_min1}; A_{s\_min2}) = 0.36 \text{ in}^2$$

Required Area of Reinforcement, $A_{sc5} = \text{MAX}(A_{s5}; A_{s\_min}) = 0.44 \text{ in}^2$

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.5

Provided Reinforcement, $A_{sb} = \text{TAB}"ACI/Bar"; Asb; Bar=Bar" = 0.31 \text{ in}^2$

Number of Bars, $n = 2$

Vertical Reinforcement, $A_{sc5\_Prov} = A_{sb} \times n = 0.62 \text{ in}^2$

Check Validity= IF($A_{sc5\_Prov} \geq A_{sc5}$; "Valid"; "Invalid") = Valid

Design Summary

Area of Reinforcement for Exterior Negative Moment of End Span: $A_{sc1\_Prov} = 0.44 \text{ in}^2$

Area of Reinforcement for Positive Moment of End Span: $A_{sc2\_Prov} = 0.62 \text{ in}^2$

Area of Reinforcement for Interior Negative Moment of End Span: $A_{sc3\_Prov} = 0.93 \text{ in}^2$

Area of Reinforcement for Interior Negative Moment of Interior Span: $A_{sc4\_Prov} = 0.93 \text{ in}^2$

Area of Reinforcement for Interior Positive Moment of Interior Span: $A_{sc5\_Prov} = 0.62 \text{ in}^2$

Interactive Design Aids for Structural Engineers
Design Two-Way Slab without Beams Analyzed by the Direct Design Method As per ACI 318-11

System

Longer Span for Two-Way Slab, \( L_n = 18.00 \) ft
Shorter Span for Two-Way Slab, \( L_s = 14.00 \) ft
Thickness of Slab, \( h = 7.00 \) in
Concrete Cover, \( c_0 = 1.25 \) in
Depth of Slab, \( d = h - c_0 \) = 5.75 in
Square Column Dimension, \( b_c = 16.00 \) in
Width of Column Strip, \( b = \frac{L_s \times 12}{2} \) = 84 in
Width of Middle Strip, \( b_m = \frac{L_s \times 12 - b}{2} \) = 84 in

Load

Slab Self Weight, \( q_s = \frac{h}{12} \times 150 \) = 87.50 psf
Partition Load, \( q_p = 20.00 \) psf
Dead Load, \( q_D = q_s + q_p \) = 107.50 psf
Live Load, \( q_L = 40.00 \) psf
Ultimate Load, \( q_U = 1.2 \times q_D + 1.6 \times q_L \) = 193.00 psf

Material Properties

Concrete Strength, \( f'_c = 3000 \) psi
Yield Strength of Reinforcement, \( f_y = 60000 \) psi

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Two-Way Slab Analyzed by the Direct Design Method

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Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = 0.90 \)

Minimum Reinforcement Ratio (According to Cl.7.12.2 of ACI318),

\[
\rho_{\text{min}} = \begin{cases} 
0.002 & \text{if } f_y \leq 50000 \\
0.0014 & \text{if } 77143 \leq f_y \leq 77143 \\
0.0018 & \text{otherwise}
\end{cases}
\]

Total Static Moment of Slab

Total Factored Static Moment Per Span (According to Eq. 13-4 of ACI318),

\[
M_0 = \frac{q_u L_s}{8 \times 1000} \left( \frac{b_c}{L_n - \frac{b_c}{12}} \right)^2 = 93.82 \text{ kip*ft}
\]

Flexural Reinforcement Required for Negative Moment of Column Strip

\[
R_{n1} = \frac{M_0 \times 12000 \times 0.53}{\Phi \times b \times d^2} = 239 \text{ psi}
\]

Ratio of RFT, \( \rho_1 = \frac{0.85 f'_c}{f_y} \left[ 1 - \sqrt{1 - \frac{2R_{n1}}{0.85 f'_c}} \right] \)

Area of Steel, \( A_{s1_{\text{req}}} = \max(\rho_1 \rho_{\text{min}}) \times b \times d = 2.02 \text{ in}^2 \)

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar=Bar)

Provided Reinforcement, \( A_{sb} = 0.44 \text{ in}^2 \)

Number of Bars, \( n = 5 \)

Vertical Reinforcement, \( A_{s1_{\text{prov}}} = A_{sb} \times n = 2.20 \text{ in}^2 \)

Check Validity= IF\( (A_{s1_{\text{prov}}}>A_{s1_{\text{req}}}; "Valid"; "Invalid") = Valid \)

Flexural Reinforcement Required for Positive Moment of Column Strip

\[
R_{n2} = \frac{M_0 \times 12000 \times 0.31}{\Phi \times b \times d^2} = 140 \text{ psi}
\]

Ratio of RFT, \( \rho_2 = \frac{0.85 f'_c}{f_y} \left[ 1 - \sqrt{1 - \frac{2R_{n2}}{0.85 f'_c}} \right] \)

Area of Steel, \( A_{s2_{\text{req}}} = \max(\rho_2 \rho_{\text{min}}) \times b \times d = 1.16 \text{ in}^2 \)

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar=Bar)

Provided Reinforcement, \( A_{sb} = 0.44 \text{ in}^2 \)

Number of Bars, \( n = 3 \)

Vertical Reinforcement, \( A_{s2_{\text{prov}}} = A_{sb} \times n = 1.32 \text{ in}^2 \)

Check Validity= IF\( (A_{s2_{\text{prov}}}>A_{s2_{\text{req}}}; "Valid"; "Invalid") = Valid \)
Flexural Reinforcement Required for Negative Moment of Middle Strip

\[
R_{n3} = \frac{M_0 \times 12000 \times 0.17}{\phi \times b \times d^2} = 77 \text{ psi}
\]

Ratio of RFT, \( \rho_3 = \frac{0.85 \times f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2 \times R_{n3}}{0.85 \times f'_c}}\right) = 0.00130\]

Area of Steel, \( A_{s3\_Req} = \text{MAX}(\rho_3 \times \rho_{3\_min}) \times b \times d = 0.87 \text{ in}^2 \)

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.6

Provided Reinforcement, \( A_{sb} = \text{TAB}("ACI/Bar"; Asb; Bar=Bar) = 0.44 \text{ in}^2 \)

Number of Bars, \( n = 2 \)

Vertical Reinforcement, \( A_{s3\_Prov} = A_{sb} \times n = 0.88 \text{ in}^2 \)

Check Validity = IF\( (A_{s3\_Prov} \geq A_{s3\_Req}) \, \text{"Valid"; "Invalid") = Valid}\)

Flexural Reinforcement Required for Positive Moment of Middle Strip

\[
R_{n4} = \frac{M_0 \times 12000 \times 0.21}{\phi \times b \times d^2} = 95 \text{ psi}
\]

Ratio of RFT, \( \rho_4 = \frac{0.85 \times f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2 \times R_{n4}}{0.85 \times f'_c}}\right) = 0.00161\]

Area of Steel, \( A_{s4\_Req} = \text{MAX}(\rho_4 \times \rho_{4\_min}) \times b \times d = 0.87 \text{ in}^2 \)

Provided Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.6

Provided Reinforcement, \( A_{sb} = \text{TAB}("ACI/Bar"; Asb; Bar=Bar) = 0.44 \text{ in}^2 \)

Number of Bars, \( n = 2 \)

Vertical Reinforcement, \( A_{s4\_Prov} = A_{sb} \times n = 0.88 \text{ in}^2 \)

Check Validity = IF\( (A_{s4\_Prov} \geq A_{s4\_Req}) \, \text{"Valid"; "Invalid") = Valid}\)

Design Summary

Area of Reinforcement Required for Negative Moment of Middle Strip, \( A_{s1\_Prov} = A_{s1\_Prov} = 2.20 \text{ in}^2 \)

Area of Reinforcement Required for Negative Moment of Middle Strip, \( A_{s2\_Prov} = A_{s2\_Prov} = 1.32 \text{ in}^2 \)

Area of Reinforcement Required for Negative Moment of Middle Strip, \( A_{s3\_Prov} = A_{s3\_Prov} = 0.88 \text{ in}^2 \)

Area of Reinforcement Required for Positive Moment of Middle Strip, \( A_{s4\_Prov} = A_{s4\_Prov} = 0.88 \text{ in}^2 \)
Calculating Development Length of Bars in Tension as per ACI 318-11 Chapter 12

Material Properties

Concrete Strength, \( f'_c = 4000 \text{ psi} \)
Yield Strength of Reinforcement, \( f_y = 60000 \text{ psi} \)
Modification Factor for Lightweight Concrete, \( \lambda = 0.75 \)

Factor of Development Length Based on RFT Location (According to Cl.12.2.4 of ACI318), \( \psi_t = 1.30 \)
Factor of Development Length Based on RFT Coating (According to Cl.12.2.4 of ACI318), \( \psi_e = 1.50 \)
Maximum Modifying Factor, \( \psi_{te} = \text{MIN}(\psi_t, \psi_e; 1.7) = 1.70 \)
Identification of, Bar= SEL("ACI/Bar" ;Bar; ) = No.7
Diameter of Bars, \( d_b = 0.88 \text{ in} \)

Calculation of Development Length

1. Class A Splice
Development Length for Bars No.6 and Smaller (According to Cl.12.2.2 of ACI318),
\[
L_{d,A1} = \left( \frac{3 \cdot f_y \cdot \psi_{te}}{50 \cdot \lambda \cdot \sqrt{f'_c}} \right) \cdot d_b = 114 \text{ in}
\]
Development Length for Bars No.7 and Greater (According to Cl.12.2.2 of ACI318),
\[
L_{d,A2} = \left( \frac{3 \cdot f_y \cdot \psi_{te}}{40 \cdot \lambda \cdot \sqrt{f'_c}} \right) \cdot d_b = 142 \text{ in}
\]
\[
L_{d,A} = \text{IF}(d_b \leq 0.75 ; L_{d,A1} ; L_{d,A2} ) = 142 \text{ in}
\]

2. Class B Splice
Development Length for Bars No.6 and Smaller (According to Cl.12.2.2 of ACI 318),
\[
L_{d,B1} = \left( \frac{3 \cdot f_y \cdot \psi_{te}}{50 \cdot \lambda \cdot \sqrt{f'_c}} \right) \cdot 1.3 \cdot d_b = 148 \text{ in}
\]
Chapter 1: Concrete Design
Development Length of Bars in Tension

Development Length for Bars No. 7 and Greater (According to Cl. 12.2.2 of ACI 318),

\[
L_{d_B2} = \left( \frac{3f_y \psi_{le}}{40 \lambda \sqrt{f'_c}} \right) \times 1.3d_b = 184 \text{ in}
\]

\[
L_{d_B} = \text{IF}(d_b \leq 0.75; L_{d_B1}; L_{d_B2}) = 184 \text{ in}
\]

Calculation Summary

Development Length for Class A, \(L_{d_A} = L_{d_A} = 142 \text{ in}\)

Development Length for Class B, \(L_{d_B} = L_{d_B} = 184 \text{ in}\)
System
Spacing between Bolts along x-x, B= 6.00 in
Spacing between Bolts along y-y, L= 6.00 in
Distance to Edge from Nearest bolt, e= 3.00 in

Load
Ultimate Load, \( P_u = \) 14000 lb
Number of Anchors, \( n = \) 4

Material Properties
Concrete Strength, \( f'_c = \) 4000 psi
Tensile Strength of Anchor Bolt Grade, \( f_{uta} = \) 60000 psi
Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), \( \phi_1 = \) 0.75
Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), \( \phi_2 = \) 0.70
Modification Factor for Lightweight Concrete, \( \lambda = \) 1.00

Determine Anchor Diameter
Required Effective Area of Anchor Bolt (According to Eq.D.2 of ACI318),
\[
A_{se,Req} = \frac{P_u}{\phi_1 \cdot n \cdot f_{uta}} = 0.078 \text{ in}^2
\]
Provided Anchor Bolt, Dia= SEL("ACI/Anchor"; Dia; ) = 0.500 in
Provided Area of Anchor Bolt, \( A_{se,Prov} = \) TAB("ACI/Anchor"; Ase; Dia=Dia) = 0.142 in^2
Check Validity= IF(\( A_{se,Prov} \geq A_{se,Req} \); "Valid"; "Increase Dia") = Valid
Determine Embedment Length

Assume that, \( h_{\text{ef,Prov}} = 4.50 \text{ in} \)

Projected Area of Failure Surface for Anchors (According to Cl.D.5.2.1 of ACI318),
\[
A_{nc} = (1.5 \cdot h_{\text{ef,Prov}} + L + e) \cdot (1.5 \cdot 2 \cdot h_{\text{ef,Prov}} + B) = 307 \text{ in}^2
\]

Projected Area of Failure Surface for Single Anchor (According to Cl.D.5.2.1 of ACI318),
\[
A_{nco} = \frac{9 \cdot h_{\text{ef,Prov}}^2}{2} = 182 \text{ in}^2
\]

Check Validity:
\[
\text{IF}(A_{nc} < n \cdot A_{nco}; \text{"Valid"}; \text{"Increase hef")}
\]

Factor (According to Cl.D.5.2.4 of ACI318), \( \psi_{ec,N} = 1.00 \)

Factor (According to Cl.D.5.2.5 of ACI318), \( \psi_{ed,N} = 0.7 + \frac{0.3 \cdot e}{1.5 \cdot h_{\text{ef,Prov}}} = 0.83 \)

Factor (According to Cl.D.5.2.6 of ACI318), \( \psi_{cp,N} = 1.00 \)

Basic Strength of Concrete Breakout (According to Eq.D-6 of ACI318),
\[
N_b = 24 \cdot \lambda \cdot \sqrt{f'c} \cdot h_{\text{ef,Prov}}^{1.5} = 14490 \text{ lb}
\]

Nominal Strength of Concrete Breakout (According to Eq.D-5 of ACI318),
\[
N_{cbg} = \frac{A_{nc}}{A_{nco}} \cdot \psi_{ec,N} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} \cdot N_b = 20287 \text{ lb}
\]

Check Validation:
\[
\text{IF}(P_u < \Phi_2 \cdot N_{cbg}; \text{"Valid"}; \text{"Increase h_{ef")})
\]

Calculation of Required Head Size

Factor (According to Cl.D.5.3.6 of ACI318), \( \psi_{c,p} = 1.00 \)

Required Head Size for Anchor Bolt (According to Eq.D-15 of ACI318),
\[
A_{brg} = \frac{P_u / n}{\Phi_2 \cdot \psi_{c,p} \cdot 8 \cdot f'c} = 0.156 \text{ in}^2
\]

Design Summary

Diameter of Anchor Bolt, Dia = 0.500 in
Embedment Length of Anchor Bolt, \( h_{\text{ef}} = h_{\text{ef,Prov}} = 4.50 \text{ in} \)
Head Size of Anchor Bolt, \( A_{brg} = A_{brg} = 0.156 \text{ in}^2 \)
Shear Strength of Slab at Column Support as per ACI 318-11 Chapter 11

System

- Width of Column, $c_1$ = 48.0 in
- Length of Column, $c_2$ = 8.0 in
- Thickness of Slab, $t$ = 10.0 in
- Concrete Cover, $c_o$ = 3.5 in
- Effective Depth of Slab, $d$ = $t - c_o$ = 6.5 in

Load

- Ultimate Shear Force, $V_u$ = 20 kips

Material Properties

- Concrete Strength, $f'_c$ = 4000 psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi$ = 0.75
- Modification Factor for Lightweight Concrete, $\lambda$ = 1.00

Check Slab Thickness

- Perimeter of Critical Section for Two-Way Shear, $b_0$ = $2 * (c_1 + d) + 2 * (c_2 + d)$ = 138 in
- Column Type = SEL("ACI/Alfa S"; Type; ) = Interior
- Alfa Constant, $\alpha_s$ = TAB("ACI/AlfaS"; Alfa; Type=Type) = 40.00
- Ratio of Long to Short Column Dimensions, $\beta$ = MAX($c_1$;$c_2$)/MIN($c_1$;$c_2$) = 6.00
- Concrete Shear Strength (According to Eq. 11-31 of ACI318), $V_{c1}$ = $(2 + 4/\beta) * \lambda * \sqrt{f'_c * b_0 * d / 1000}$ = 151.3 kips
- Concrete Shear Strength (According to Eq. 11-32 of ACI318), $V_{c2}$ = $(\alpha_s * d / b_0 + 2) * \lambda * \sqrt{f'_c * b_0 * d / 1000}$ = 220.3 kips
Concrete Shear Strength (According to Eq. 11-33 of ACI318),

\[ V_{c3} = 4\lambda \sqrt{f'_{c}} b_0 d / 1000 \]

= 226.9 kips

Nominal Concrete Shear Strength, \( \Phi V_c = \Phi \times \text{MIN}(V_{c1}; V_{c2}; V_{c3}) \)

= 113.5 kips

Validation = IF( \( \Phi V_c > V_u \); "O.K."; "Increase Depth" )

= O.K.

**Calculation Summary**

Thickness of Slab, \( t \) = 10 in
Design of Simple Span Deep Beam by the Strut-and-Tie Model as per ACI318 Appendix A

![Diagram of a Simple Span Deep Beam](image)

**System**

- Width of Deep Beam, $b = 7.0$ in
- Height of Deep Beam, $h = 60.0$ in
- Concrete Cover, $c_o = 1.25$ in
- Depth of Deep Beam, $d = h - c_o = 58.75$ in
- End Distance of Truss Model, $x = 5.0$ in
- Span of Deep Beam, $L_n = 13.3$ ft
- Column Width, $b_c = 20.0$ in

**Load**

- Dead Load for Column, $P_D = 173.35$ kips
- Live Load for Column, $P_L = 270.0$ kips
- Service Load for Column, $P = 1.0 \times P_D + 1.0 \times P_L = 443.4$ kips
- Ultimate Load for Column, $P_u = 1.2 \times P_D + 1.6 \times P_L = 640.0$ kips
Material Properties

Concrete Strength, $f'_c = 4000$ psi

Yield Strength of Reinforcement, $f_y = 60000$ psi

Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi = 0.75$

Modification Factor for Lightweight Concrete, $\lambda = 1.00$

Friction Factor (According to Cl.11.6.4.3 of ACI318), $\mu = 1.4 \ast \lambda = 1.40$

Check Deep Beam Requirements

Check on Height of Deep Beam Requirements (According to Cl.11.7.1 of ACI318),

$$R = \text{IF}(12 \ast Ln/h < 4; \text{"Deep Beam Design"}; \text{"Normal Beam Design"}) = \text{Deep Beam Design}$$

Estimation of Truss Model

Length of Diagonal Strut, $L_1 = \sqrt{\left(\frac{Ln \ast 12}{2}\right)^2 + \left(h - 2 \ast x\right)^2} = 94.17$ in

The Force in Diagonal Strut, $F_s = \frac{P_u \ast L_1}{2 \ast h - 2 \ast x} = 602.69$ kips

The Force in Horizontal Tie, $F_t = \frac{P_u \ast 0.5 \ast Ln \ast 12}{2 \ast h - 2 \ast x} = 510.72$ kips

Angle Between Diagonal Strut and Horizontal Tie, $\alpha = \text{atan}\left(\frac{h - 2 \ast x}{0.5 \ast Ln \ast 12}\right) = 32.07^\circ$

Check Validity (According to Cl.A.2.5 of ACI318) = IF($\alpha > 25$; "Valid"; "Invalid") = Valid

Calculation of Effective Concrete Strength

(According to Cl.3.2.2(a) of ACI318) Factor of, $\beta_s = 0.75$

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

$$f_{ce1} = 0.85 \ast \beta_s \ast f'_c = 2550$$ psi

Calculation of Effective Concrete Strength for Nodal Zones

For Nodal Zone C Bounded by Three Struts (C-C-C Nodal Zone)

(According to Cl.A.5.2.1 of ACI318) Factor of, $\beta_n = 1.00$

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

$$f_{ce2} = 0.85 \ast \beta_n \ast f'_c = 3400$$ psi

For Nodal Zone A&B Bounded by Three Struts (C-C-T Nodal Zone)

(According to Cl.A.5.2.2 of ACI318) Factor of, $\beta_n = 0.80$

Effective Concrete Strength (According to Eq.A-3 of ACI 318),

$$f_{ce3} = 0.85 \ast \beta_n \ast f'_c = 2720$$ psi

Minimum Effective Concrete Strength, $f_{ce} = \text{MIN}(f_{ce1}; f_{ce2}; f_{ce3}) = 2550$ psi
Check Strength at Node C

The Length of The Horizontal Face of Nodal Zone C,

\[ L_{hc} = \frac{P_u \times 1000}{\Phi b_c f_{ce}} = 16.73 \text{ in} \]

The Length of Other Faces of Nodal Zone C,

\[ L_c = \frac{L_{hc} F_s}{P_u} = 15.75 \text{ in} \]

Check Strength at Node A&B

The Length of The Horizontal Face of Nodal Zone A,

\[ L_{ha} = \frac{F_t \times 1000}{\Phi b_c f_{ce}} = 13.35 \text{ in} \]

Width of Node at Support A,

\[ L_a = \frac{0.5 P_u \times 1000}{\Phi b_c f_{ce}} = 8.37 \text{ in} \]

Calculation VL. and HZ. Reinforcement to Resist Splitting Diagonal Struts

1. Vertical Reinforcement

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.4
Provided Reinforcement, \( A_{sbv} = \), TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in²
Number of Bars, \( n_v = \) 4
Vertical Reinforcement, \( A_{sv} = \), \( A_{sbv} \times n_v = 0.80 \text{ in}^2 \)
Provided Spacing between Bars, \( s = 11.00 \text{ in} \)
Vertical Reinforcement (According to Eq.A4 of ACI318),

\[ V_L = \frac{A_{sv}}{b_c \times s} \times \sin(90 - \alpha) = 0.00308 \]

2. Horizontal Reinforcement

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.5
Provided Reinforcement, \( A_{sbh} = \), TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in²
Number of Bars, \( n_h = \) 2
Vertical Reinforcement, \( A_{sh} = \), \( A_{sbh} \times n_h = 0.62 \text{ in}^2 \)
Provided Spacing between Bars, \( s = 11.00 \text{ in} \)
Horizontal Reinforcement (According to Eq.A4 of ACI318),

\[ H_Z = \frac{A_{sh}}{b_c \times s} \times \sin(\alpha) = 0.00150 \]

Check Validity= IF(VL+HZ>0.003; "Valid"; "Invalid") = Valid
Calculation of Tension Reinforcement for Tie Connecting Node A&B

\[
\text{Required Reinforcement Area, } A_{\text{req}} = \frac{Ft \times 1000}{\phi \times f_y} = 11.35 \text{ in}^2
\]

\[
\text{Provided Reinforcement, } \text{Bar} = \text{SEL("ACI/Bar"; Bar; )} = \text{No.8}
\]

\[
\text{Provided Reinforcement, } A_{s_{\text{b}}} = \text{TAB("ACI/Bar"; Asb; Bar=Bar)} = 0.79 \text{ in}^2
\]

\[
\text{Number of Bars, } n = 16
\]

\[
\text{Total Provided Area, } A_{s_{\text{prov}}} = n \times A_{s_{\text{b}}} = 12.64 \text{ in}^2
\]

\[
\text{Check Validity} = \text{IF(Aprov>Areq; "Valid"; "Invalid")} = \text{Valid}
\]

Design Summary

\[
\text{Provided Vertical Reinforcement, } A_{s_{v}} = A_{s_{v}} = 0.80 \text{ in}^2
\]

\[
\text{Provided Horizontal Reinforcement, } A_{s_{h}} = A_{s_{h}} = 0.62 \text{ in}^2
\]

\[
\text{Provided Tension Reinforcement, } A_{s_{prov}} = A_{s_{prov}} = 12.64 \text{ in}^2
\]
Design of Continuous Deep Beam by the Strut-and-Tie Model as per ACI318 Appendix A

System

Width of Deep Beam, \( b = \) 24.0 in
Height of Deep Beam, \( h = \) 144.0 in
Concrete Cover, \( c_o = \) 1.25 in
Depth of Deep Beam, \( d = \) \( h - c_o \) = 142.75 in
Upper End Distance of Truss Model, \( x_1 = \) 6.0 in
Lower End Distance of Truss Model, \( x_2 = \) 9.0 in
Span of Deep Beam, \( L_n = \) 24.0 ft
Exterior Planted Column Width, \( b_{c1} = \) 24.0 in
Interior Planted Column Width, \( b_{c2} = \) 56.0 in
Distance between Supports of Deep Beam, \( L_s = \) 24.0 ft
Support Column Width, \( b_s = \) 48.0 in
Support Column Depth, \( d_s = \) 24.0 in

Load

Dead Load for Exterior Column, \( P_{D1} = \) 100.0 kips
Live Load for Exterior Column, \( P_{L1} = \) 237.5 kips
Ultimate Load for Exterior Column, \( P_{u1} = 1.2 \times P_{D1} + 1.6 \times P_{L1} = \) 500.0 kips
Dead Load for Interior Column, \( P_{D2} = \) 750.0 kips
Live Load for Interior Column, \( P_{L2} = \) 1000.0 kips
Ultimate Load for Interior Column, \( P_{u2} = 1.2 \times P_{D2} + 1.6 \times P_{L2} = \) 2500.0 kips
Support Column Ultimate Load, \( P_u = P_{u1} + P_{u2} / 2 = \) 1750.0 kips

Interactive Design Aids for Structural Engineers
Material Properties

Concrete Strength, $f'_c = 4000$ psi
Yield Strength of Reinforcement, $f_y = 60000$ psi
Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi = 0.75$
Modification Factor for Lightweight Concrete, $\lambda = 1.00$
Friction Factor (According to Cl.11.6.4.3 of ACI318), $\mu = 1.4 \times \lambda = 1.40$

Check Deep Beam Requirements

Check on Height of Deep Beam Requirements (According to Cl.11.7.1 of ACI318),
\[ R = IF(12*Ln/h<4; \text{"Deep Beam Design"}; \text{"Normal Beam Design"}) = \text{Deep Beam Design} \]

Calculation of Effective Concrete Strength

(According to Cl.A.3.2 of ACI318) Factor of, $\beta_s = 1.00$
Effective Concrete Strength (According to Eq.A-3 of ACI 318),
\[ f_{ce1} = 0.85 \times \beta_s \times f'_c = 3400 \text{ psi} \]

Calculation of Effective Concrete Strength for Nodal Zones

For Nodal Zone IV Bounded by Three Struts (C-C-C Nodal Zone)
(According to Cl.A.5.2.1 of ACI318) Factor of, $\beta_n = 1.00$
Effective Concrete Strength (According to Eq.A-3 of ACI 318),
\[ f_{ce2} = 0.85 \times \beta_n \times f'_c = 3400 \text{ psi} \]

For Nodal Zone A&B Bounded by Three Struts (C-C-T Nodal Zone)
(According to Cl.A.5.2.2 of ACI318) Factor of, $\beta_n = 0.80$
Effective Concrete Strength (According to Eq.A-3 of ACI 318),
\[ f_{ce3} = 0.85 \times \beta_n \times f'_c = 2720 \text{ psi} \]
Minimum Effective Concrete Strength, $f_{ce} = \text{MIN}(f_{ce1}; f_{ce2}; f_{ce3}) = 2720 \text{ psi}$

Calculation of Forces in Struts

For Node IV Will Carry Exterior Column Load Strut, $F_a = P_{u1} = 500.00$ kips
For Node IV Other Struts B and C, $F_{bc} = 0.5 \times (P_u - F_a) = 625.00$ kips

Check Width of Struts at Node IV

Width of Strut a, $W_{sa} = \frac{F_a \times 1000}{\Phi \times f_{ce} \times b} = 10.21$ in
Width of Strut b&c, $W_{sbc} = \frac{F_{bc} \times 1000}{\Phi \times f_{ce} \times b} = 12.77$ in
Total Width of Struts, $W_s = W_{sa} + W_{sbc} \times 2 = 35.75$ in
Check Validity= \[ IF(W_s < b_s; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \]
Check Width of Struts at Node I

Width of Strut, $W_{sI} = \frac{P_{u1} \times 1000}{\phi \times f_{ce} \times b} = 10.21$ in

Check Validity = IF($W_{sI} < b_{c1}$; "Valid"; "Invalid") = Valid

Check Width of Struts at Node II

Width of Strut, $W_{sII} = \frac{P_{u2} \times 1000}{\phi \times f_{ce} \times b} = 51.06$ in

Check Validity = IF($W_{sII} < b_{c2}$; "Valid"; "Invalid") = Valid

Calculation of Force in Strut I-IVA and Tie I-IIA

Horizontal Projection of Strut I-IVA, $L_{hiiva} = \frac{(L_n^2 - L_s^2) \times 12}{2} - W_{sbc} = 131.23$ in

Vertical Projection of Strut I-IVA, $L_{viiva} = h - (x_1 + x_2) = 129.00$ in

Horizontal Force in Strut I-IVA and Tie I-IIA, $F_{iiva} = \frac{P_{u1} \times L_{hiiva}}{L_{viiva}} = 508.64$ kips

Length of Strut I-IVA, $L_{iiva} = \sqrt{L_{hiiva}^2 + L_{viiva}^2} = 184.02$ in

Compression Force in Strut I-IVA at Node I, $F_i = \frac{P_{u1} \times L_{iiva}}{h - (x_1 + x_2)} = 713.26$ kips

Check Validity = IF($F_i < f_{ce}$; "Valid"; "Invalid") = Valid

Calculation of Width of Strut IIA-IVB

Horizontal Projection of Strut IIA-IVB, $L_{hiiaivb} = \frac{(L_n^2 - L_s^2) \times 12}{2} \times W_{sII} \times 3}{8} = 124.9$ in

Vertical Projection of Strut IIA-IVB, $L_{viiaivb} = h - (x_1 + x_2 \times 2) = 120.0$ in

Vertical Force in Strut IIA-IVB, $F_{iiiaivb} = \frac{L_{hiiaivb} \times F_{iiva}}{L_{viiaivb}} = 529.4$ kips

Calculation of Width of Strut IIA-IVC

Horizontal Projection of Strut IIA-IVB, $L_{hiiaivc} = \frac{(L_n^2 - L_s^2) \times 12}{2} \times W_{sII} \times 3}{8} = 124.9$ in

Vertical Projection of Strut IIA-IVB, $L_{viiaivc} = h - (x_1 + x_2 \times 2) = 120.0$ in

Vertical Force in Strut IIA-IVB, $F_{iiiaivc} = \frac{L_{hiiaivc} \times F_{iiva}}{L_{viiaivc}} = 529.4$ kips
Calculation of Width of Strut IIb-IIVc

Horizontal Projection of Strut IIa-IIVb, Lhiaivb=
\[
\frac{(L_n \cdot 2 - L_s) \cdot 12}{2} - \frac{W_{II} \cdot 7}{50} = 136.9 \text{ in}
\]

Vertical Projection of Strut IIa-IIVb, Lvialvb=
\[
h - (x_1 + x_2 \cdot 2) = 120.0 \text{ in}
\]

Vertical Force in Strut IIa-IIVb, Fiiavb=
\[
\frac{L_{hiaivb}}{L_{vialvb}} \cdot F_i = 813.7 \text{ kips}
\]

Calculation of Width of Tie IVc-Va

Force in Tie IVc-Va, Fivcva=
\[
\frac{F_{iiavb} \cdot 1000}{\phi \cdot f_{ce} \cdot b} = 16.62 \text{ in}
\]

Calculation VL. and HZ. Reinforcement to Resist Splitting of Diagonal Struts

1. Vertical Reinforcement
   - Angle of Strut, \( \alpha = 46.60^\circ \)
   - Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.5
   - Provided Reinforcement, \( A_{sbv} = \) TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in²
   - Number of Bars, \( n_v = 2 \)
   - Vertical Reinforcement, \( A_{sv} = A_{sbv} \cdot n_v = 0.62 \text{ in}^2 \)
   - Provided Spacing between Bars, \( s = 10.00 \text{ in} \)
   - Vertical Reinforcement (According to Eq.A4 of ACI318),
     \[
     VL = \frac{A_{sv} \cdot \sin(90 - \alpha)}{b \cdot s} = 0.00177
     \]

2. Horizontal Reinforcement
   - Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.5
   - Provided Reinforcement, \( A_{sbh} = \) TAB("ACI/Bar"; Asb; Bar=Bar) = 0.31 in²
   - Number of Bars, \( n_h = 2 \)
   - Vertical Reinforcement, \( A_{sh} = A_{sbh} \cdot n_h = 0.62 \text{ in}^2 \)
   - Provided Spacing between Bars, \( s = 10.00 \text{ in} \)
   - Horizontal Reinforcement (According to Eq.A4 of ACI318),
     \[
     HZ = \frac{A_{sh} \cdot \sin(\alpha)}{b \cdot s} = 0.00188
     \]

Check Validity= IF(VL+HZ>0.003; "Valid"; "Invalid") = Valid
### Calculation of Tension Reinforcement for Tie Connecting Joint I-IIa

Required Reinforcement Area, \( A_{sreq} = \frac{F_{iiva} \times 1000}{\phi \times f_y} \) = 11.30 in\(^2\)

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.9

Number of Bars, \( n = \) 12.00

Provided Reinforcement, \( A_{sb} = \) TAB("ACI/Bar"; Asb; Bar=Bar) = 1.00 in\(^2\)

Total Provided Area, \( A_{sprov} = n \times Asb \) = 12.00 in\(^2\)

Check Validity= IF(Asprov>Asreq; "Valid"; "Invalid") = Valid

### Design Summary

Provided Vertical Reinforcement, \( A_{sv} = A_{sv} \) = 0.62 in\(^2\)

Provided Horizontal Reinforcement, \( A_{sh} = A_{sh} \) = 0.62 in\(^2\)

Provided Tension Reinforcement, \( A_{sprov} = A_{sprov} \) = 12.00 in\(^2\)
Design for Transfer of Horizontal Force at Base of Column where The Footing Surface is not Intentionally Roughened as per ACI 318-11 Chapter 12

System
- Column Width, $b_c = 12.0$ in
- Column Depth, $d_c = 12.0$ in
- Footing Width, $B_f = 9.0$ ft
- Footing Length, $L_f = 9.0$ ft
- Footing Thickness, $T_f = 22.0$ in

Load
- Ultimate Horizontal Force at the Base of Column, $V_u = 84.0$ kips

Material Properties
- Concrete Strength, $f_{c'} = 4000$ psi
- Yield Strength of Reinforcement, $f_y = 60000$ psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\phi_1 = 0.75$
- Modification Factor for Lightweight Concrete, $\lambda = 1.00$
- Friction Factor (According to Cl.11.6.4.3 of ACI318), $\mu = 0.6 \times \phi_1 = 0.60$

Check on Maximum Shear Transfer Permitted
- Nominal Shear Force (According to Cl.11.6.5 of ACI318),
  \[ \Phi V_{n1} = \phi \times \left( \frac{0.2 \times f_{c'} \times 1000 \times b_c \times d_c}{1000} \right) = 86.4 \text{ kips} \]
  \[ \Phi V_{n2} = \phi \times \left( \frac{800 \times b_c \times d_c}{1000} \right) = 86.4 \text{ kips} \]
- Minimum Nominal Shear, $\Phi V_n = \text{MIN}(\Phi V_{n1}, \Phi V_{n2}) = 86.4$ kips
- Check Validity = IF($V_u < \Phi V_n$, "Valid", "Increase Dimension") = Valid
Chapter 1: Concrete Design
Transfer of Horizontal Force at Base of Column

Required Area of Reinforcement (According to Eq.11-25 of ACI318),

\[ A_{vf} = \frac{Vu \cdot 1000}{\phi \cdot f_y \cdot \mu} = 3.11 \text{ in}^2 \]

Provided Shear Reinforcement, Bar = SEL("ACI/Bar"; Bar; ) = No.8
Diameter of Bars, Dia = TAB("ACI/Bar"; Dia; Bar=Bar) = 1.0000 in
Number of Bars, n = 4

Provided Area of Reinforcement, \( A_s = \frac{n \cdot \pi \cdot \text{Dia}^2}{4} \) = 3.14 in²

Check Validity = IF(\( A_s > A_{vf} \); "Valid"; "Increase RFT") = Valid

Check on Development Length of Tensile Reinforcement with Column

Clear Cover to Center of Bars, c = 3.25 in
Center to Center Bar Spacing, S = 4.50 in
Factor of, \( cb = \text{MIN}(c + \text{Dia}/2; S/2) = 2.25 \text{ in} \)

(According to Cl.12.2.3 of ACI318) Factor of, \( K_t = 0.00 \)
(According to Cl.12.2.4 of ACI318) Factor of, \( \Psi_t = 1.00 \)
(According to Cl.12.2.4 of ACI318) Factor of, \( \Psi_e = 1.00 \)
(According to Cl.12.2.4 of ACI318) Factor of, \( \Psi_s = 1.00 \)

Development Length within Column

Development Length (According to Eq.12-1 of ACI318),

\[ L_{d1} = \frac{3 \cdot f_y \cdot \Psi_t \cdot \Psi_e \cdot \Psi_s \cdot \text{Dia}}{40 \cdot \lambda \cdot \sqrt{f_c} \cdot (cb + K_t) / \text{Dia}} = 31.6 \text{ in} \]

Development Length within Footing

Development Length (According to Cl.12.5.2 of ACI318),

\[ L_{d2} = \frac{0.02 \cdot \Psi_e \cdot f_y}{\lambda \cdot \sqrt{f_c} \cdot \text{Dia}} = 19.0 \text{ in} \]

Design Summary

Provided Area of Reinforcement, \( A_s = A_s = 3.14 \text{ in}^2 \)
Development Length within Column, \( L_{d1} = L_{d1} = 31.6 \text{ in} \)
Development Length within Footing, \( L_{d2} = L_{d2} = 19.0 \text{ in} \)

Interactive Design Aids for Structural Engineers
Design of Bearing Wall by Empirical Method as per ACI 318-11 Chapters 10 & 14

**System**
- Height of Wall, $L$ = 15 ft
- Spacing of Wall Panels, $s_p$ = 8 ft
- Width of Stem for Bearing Wall, $b_w$ = 7 in

**Load**
- Service Dead Load, $P_D$ = 28 kips
- Service Live Load, $P_L$ = 14 kips
- Ultimate Load, $P_u$ = $1.2 P_D + 1.6 P_L$ = 56 kips

**Material Properties**
- Concrete Strength, $f'_{c}$ = 4000 psi
- Bearing Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi$ = 0.65
- Modification Factor for Lightweight Concrete, $\lambda$ = 1.00

**Determine Wall Thickness**
- Assume Wall Thickness, $h$ = 7.5 in
- Minimum Wall Thickness, $h_{\text{min}}$ = MAX($L \times 12/25 \ ; 4$) = 7.2 in
- Check Validation = IF ($h > h_{\text{min}}$; "O.K.", "Increase Thickness") = O.K.
Check Concrete Bearing Strength

Loaded Area, \( A_1 = h \times b_w \) = 52.50 in\(^2\)

Nominal Concrete Bearing Capacity (According to Cl.10.14.1 of ACI318),

\[ \Phi V_b = \Phi \times 0.85 \times f'_c \times A_1 / 1000 \] = 116 kips

Check Validation = IF( \( \Phi V_b > P_u \); "Valid."); "Invalid" ) = Valid.

Calculate Design Strength of Wall

Effective Width of Wall, \( w = \min(b_w + 4h; s_p \times 12) \) = 37 in

Wall Resistant Type,
Type-1: Restrained Rotation - One or Both Ends (T/B/Both)
Type-2: Unrestrained Rotation at Both Ends
Type-3: For Walls not Braced Against Lateral Translation

Type= SEL("ACI/K" ; Type; ) = Type-1
Effective Length Factor, \( K = TAB("ACI/K" ; K ; Type=Type ) \) = 0.80

Nominal Strength of Wall (According to Eq.14-1 of ACI 318),

\[ \Phi P_n = 0.55 \times \Phi \times f'_c \times w \times h \times \left( \frac{K \times L \times 12}{32 \times h} \right)^2 \times 1000 \] = 254 kips

Check Validity= IF( \( \Phi P_n > P_u \); "Valid" ; "Invalid" ) = Valid

Determine Single Layer of Reinforcement

Vertical Area of Reinforcement for Wall (According to Cl.14.3.2 of ACI318),

\( A_{sv} = 0.0012 \times 12 \times h \) = 0.108 in\(^2\)/ft

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.4
Provided Reinforcement, \( A_{sb} = TAB("ACI/Bar"; Asb; Bar=Bar) \) = 0.20 in\(^2\)
Bar Spacing, \( s = 18 \) in

\( A_{sv_{Prov}} = A_{sb} \times 12 / s \) = 0.13 in\(^2\)

Check Validity= IF( \( A_{sv_{Prov}} > A_{sv} \); "Valid" ; "Invalid" ) = Valid

Horizontal Area of Reinforcement for Wall (According to Cl.14.3.3 of ACI318),

\( A_{sh} = 0.0020 \times 12 \times h \) = 0.180 in\(^2\)/ft

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.4
Provided Reinforcement, \( A_{sb} = TAB("ACI/Bar"; Asb; Bar=Bar) \) = 0.20 in\(^2\)
Bar Spacing, \( s = 12 \) in

\( A_{sh_{Prov}} = A_{sb} \times 12 / s \) = 0.20 in\(^2\)

Check Validity= IF( \( A_{sh_{Prov}} > A_{sh} \); "Valid" ; "Invalid" ) = Valid
Design Summary

Wall Thickness, $h = 8$ in
Length of Footing, $L = 15$ ft
Vertical Area of Reinforcement for Wall, $A_{sv, prov} = 0.13$ in$^2$
Horizontal Area of Reinforcement for Wall, $A_{sh, prov} = 0.20$ in$^2$
Shear Design of Wall as per ACI 318-11 Chapter 11

System

Height of Wall, \( h_w = \) 12.0 ft
Width of Wall, \( L_w = \) 8.0 ft
Over All Wall Depth, \( d = 0.8 \times L_w \) = 6.4 ft
Thickness of Wall, \( h = \) 8.0 in
Concrete Cover, \( c_o = \) 2.0 in
Effective Depth of Wall Section, \( d_c = h - c_o = 8.0 - 2.0 = 6.0 \) in

Load

Ultimate Bending Moment, \( M_u = \) 19200 kip*ft
Ultimate Shear Force, \( V_u = \) 200 kips
Ultimate Normal Force, \( N_u = \) 0 kips

Material Properties

Concrete Strength, \( f'_c = \) 3000 psi
Yield Strength of Reinforcement, \( f_y = \) 60000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \phi = \) 0.75
Modification Factor for Lightweight Concrete, \( \lambda = \) 1.00
Check Shear Reinforcement Requirement

Maximum Shear Strength of Wall (According to Cl.11.9.3 of ACI318),

\[ \phi V_n = \phi \cdot 10 \cdot \sqrt{f_c} \cdot h \cdot d \cdot 12 \cdot 1000 / 1000 = 252 \text{ kips} \]

Check Validity:

\[ \text{IF}(V_u \leq \phi V_n; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \]

Critical Section of Shear Force, \( L_c = \text{MIN}(L_w/2; h_w/2) \)

\[ = 4.00 \text{ ft} \]

Concrete Shear Strength (According to Eq.11-27 of ACI318),

\[ V_{c1} = 3.3 \cdot \lambda \cdot \sqrt{f_c} \cdot h \cdot d \cdot 12 \cdot N_u \cdot d / 4 \cdot L_w = 111 \text{ kips} \]

Concrete Shear Strength (According to Eq.11-28 of ACI318),

\[ V_{c2} = \left( 0.6 \cdot \lambda \cdot \sqrt{f_c} + \frac{0.2 \cdot N_u}{L_w \cdot 12 / 2} \right) \cdot \frac{h \cdot d \cdot 12}{1000} = 104 \text{ kips} \]

Concrete Shear Strength, \( V_c = \text{MIN}(V_{c1}; V_{c2}) = 104 \text{ kips} \)

Shear Reinforcement:

\[ \text{IF}(V_u < \phi \cdot V_c/2; \text{"Not Required"}; \text{"Required"}) = \text{Required} \]

Determine Horizontal Shear Reinforcement

Identification of, Bar= SEL("ACI/Bar"; Bar; ) = No.4

Provided Reinforcement, \( A_{sb} = \text{TAB}("ACI/Bar"; Asb; Bar=Bar) \)

\[ = 0.20 \text{ in}^2 \]

Number of Bars, \( n = 2 \)

Area of Shear Reinforcement, \( A_h = n \cdot A_{sb} \)

\[ = 0.40 \text{ in}^2 \]

Spacing between Bars (According to Eq.11-29 of ACI318),

\[ s_{hi} = \frac{\phi \cdot f_y / 1000 \cdot d \cdot 12 \cdot A_h}{L_w - \phi \cdot V_c} = 11.3 \text{ in} \]

Provided Reinforcement Spacing, \( s_h = 10 \text{ in} \)

Check Validity:

\[ \text{IF}(s_h \leq s_{hi}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \]

Ratio of Horizontal Shear Reinforcement (According to Cl.11.9.9.2 of ACI318),

\[ \rho_{hi} = \frac{A_h}{h \cdot s_h} = 0.005 \]

\[ \rho_h = \text{MAX}(\rho_{hi}; 0.0025) = 0.005 \]
Calculation of Vertical Shear Reinforcement

Ratio of Vertical Shear Reinforcement (According to Eq.11-30 of ACI318),

\[ \rho_{vi} = 0.0025 + 0.5 \left( \frac{h_w}{L_w} \right) \times (\rho_h - 0.0025) = 0.0037 \]

\[ \rho_v = \text{MAX}(\rho_{vi} ; 0.0025) = 0.0037 \]

Identification of, Bar= SEL("ACI/Bar" ;Bar; ) = No.4

Provided Reinforcement, \( A_{sb} = \) TAB("ACI/Bar"; Asb; Bar=Bar) = 0.20 in\(^2\)

Number of Bars, \( n = 2 \)

Area of Shear Reinforcement, \( A_v = n \times A_{sb} = 0.40 \text{ in}^2 \)

Spacing between Bars (According to Eq.11-29 of ACI318),

\[ s_{vi} = \frac{A_v}{\rho_v \times h} = \frac{0.40}{0.0037 \times 8.0} = 13.5 \text{ in} \]

Provided Reinforcement Spacing, \( s_v = 13 \text{ in} \)

Check Validity= IF\((s_v \leq s_{vi}; "Valid"; "Invalid") = Valid \)

Design Summary

Horizontal Shear Reinforcement, \( A_h = A_h = 0.40 \text{ in}^2 \)

Spacing Between Horizontal Shear Reinforcement, \( s_h = s_h = 10 \text{ in} \)

Vertical Shear Reinforcement, \( A_v = A_v = 0.40 \text{ in}^2 \)

Spacing Between Horizontal Shear Reinforcement, \( s_v = s_v = 13 \text{ in} \)
Design of Shear Friction as per ACI 318-11 Chapter 11

System

- Width of Steel Plate, $B =$ 2.00 in
- Length of Steel Plate, $L =$ 4.00 in
- Thickness of Steel Plate, $t =$ 0.25 in
- Identification of Bar, $SEL("ACI/Bar" ;Bar ; ) =$ No.3
- Diameter of Bars, $d_b =$ 0.38 in
- Number of Bars, $n =$ 2

Load

- Ultimate Shear Force, $V_u =$ 3570 lb

Material Properties

- Concrete Strength, $f'_c =$ 4000 psi
- Yield Strength of Reinforcement, $f_y =$ 60000 psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), $\Phi =$ 0.75
- Modification Factor for Lightweight Concrete, $\lambda =$ 0.75
- Friction Factor (According to Cl.11.6.4.3 of ACI318), $\mu =$ 0.7 \* $\lambda =$ 0.525
Calculation of Required Reinforcement Area

Area of Shear Friction Reinforcement (According to Eq.11-25 of ACI318),

\[ A_{vf} = \frac{V_u}{\Phi \cdot f_y \cdot \mu} = 0.151 \text{ in}^2 \]

Provided Area, \( A_{act} = \) \( n \cdot \pi \cdot d_b^2 \cdot 4 \) = 0.23 \text{ in}^2

Check Validity= IF( \( A_{act} > A_{vf} \), "Valid" ; "Invalid" ) = Valid

Design Summary

Provided Area of Reinforcement, \( A_{act} = A_{act} = 0.23 \text{ in}^2 \)
Design a Single Adhesive Anchor in Tension Away from Edges as per ACI 318-11 Appendix D

System

Diameter of Adhesive Anchor Bolt, Dia = SEL("ACI/Anchor": Dia; ) = 0.500 in
Area of Adhesive Anchor Bolt, A_{se,N} = TAB("ACI/Anchor": Ase; Dia=Dia) = 0.142 in^2
Effective Embedment Length, h_{ef} = 4.0 in

Material Properties

Concrete Strength, f'_c = 4000 psi
Characteristic Bond Stress in Cracked Concrete, \( \tau_{cr} \) = 300 psi
Characteristic Bond Stress in Uncracked Concrete, \( \tau_{uncr} \) = 1000 psi
Tensile Strength of Anchor Bolt Grade, f_{uta} = 58000 psi
Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), \( \Phi_1 \) = 0.75
Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), \( \Phi_2 \) = 0.45
Modification Factor for Lightweight Concrete, \( \lambda \) = 1.00

Determine The Steel Strength of Adhesive Anchor

The Steel Strength of Anchor Bolt (According to Cl.D.4.1.1 of ACI318),

\[ \Phi N_{sa} = \Phi_1 A_{se,N} f_{uta} \]

\[ = 6177 \text{ lb} \]
Determine The Bond Strength of Adhesive Anchor

(According to Eq.D-21 of ACI318),
\[ c_{Na} = 10 \times \text{Dia} \times \sqrt{\tau_{uncr} / 1100} \]
\[ = 4.77 \text{ in} \]

(According to Eq.D-20 of ACI318),
\[ A_{Nao} = (2 \times c_{Na})^2 \]
\[ = 91.0 \text{ in}^2 \]

(According to Cl.D.5.5.1 of ACI318),
\[ A_{Na} = A_{Nao} \]
\[ = 91.0 \text{ in}^2 \]

The Basic Bond Strength (According to Eq.D-22 of ACI318),
\[ N_{ba} = \lambda \times \tau_{cr} \times \pi \times \text{Dia} \times h_{ef} \]
\[ = 1885 \text{ lb} \]

Factor (According to Cl.D.5.5.3 of ACI318),
\[ \psi_{ed,Na} = 1.00 \]

Factor (According to Cl.D.5.5.5 of ACI318),
\[ \psi_{c,Na} = 1.00 \]

The Basic Bond Strength for A Single Anchor (According to Eq.D-3 of ACI318),
\[ \Phi N_a = \Phi_2 (A_{Na}/A_{Nao}) \times \psi_{ed,Na} \times \psi_{c,Na} \times N_{ba} \]
\[ = 848 \text{ lb} \]

Determine The Concrete Breakout Strength

(According to Eq.D-6 of ACI318),
\[ \kappa_c = 17.0 \]

Basic Strength of Concrete Breakout (According to Eq.D-6 of ACI318),
\[ N_a = \kappa_c \times \lambda \times \sqrt{f'_c} \times h_{ef}^{1.5} \]
\[ = 8601 \text{ lb} \]

Factor (According to Cl.D.5.2.6 of ACI318),
\[ \psi_{cp,Na} = 1.00 \]

The Strength of Concrete Breakout (According to Eq.D-3 of ACI318),
\[ \Phi N_{cb} = \Phi_2 (A_{Na}/A_{Nao}) \times \psi_{ed,Na} \times \psi_{c,Na} \times \psi_{cp,Na} \times N_a \]
\[ = 3870 \text{ lb} \]

Determine The Tension Force Carried by Adhesive Anchor Bolt

The Tension Force Carried by Adhesive Anchor, \[ T_u = \text{MIN}(\Phi N_{sa}; \Phi N_a; \Phi N_{cb}) \]
\[ = 848 \text{ lb} \]

Design Summary

The Steel Strength of Adhesive Anchor Bolt, \[ \Phi N_{sa} = \Phi N_{sa} \]
\[ = 6177 \text{ lb} \]

The Bond Strength of Adhesive Anchor Bolt, \[ \Phi N_a = \Phi N_a \]
\[ = 848 \text{ lb} \]

The Concrete Breakout Strength of Adhesive Anchor Bolt, \[ \Phi N_{cb} = \Phi N_{cb} \]
\[ = 3870 \text{ lb} \]

The Tension Force Carried by Adhesive Anchor, \[ T_u = T_u \]
\[ = 848 \text{ lb} \]
Design a Single Headed Anchor Bolt in Tension Away from Edges as per ACI 318-11 Appendix D

Load
Ultimate Load, $P_u =$ 7000 lb

Material Properties
Concrete Strength, $f'_c =$ 4000 psi
Tensile Strength of Anchor Bolt Grade, $f_{uta} =$ 58000 psi
Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), $\Phi_1 =$ 0.75
Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), $\Phi_2 =$ 0.70
Modification Factor for Lightweight Concrete, $\lambda =$ 1.00

Determine Anchor Diameter
Required Effective Area of Anchor Bolt (According to Eq.D.3 of ACI318),

$$A_{se\_Req} = \frac{P_u}{\Phi_1*1.0*f_{uta}} = 0.161 \text{ in}^2$$

Provided Anchor Bolt, Dia= SEL("ACI/Anchor”; Dia; ) = 0.625 in
Provided Area of Anchor Bolt, $A_{se\_Prov} =$ TAB("ACI/Anchor"; Ase; Dia=Dia) = 0.226 in$^2$
Check Validity= IF($A_{se\_Prov} \geq A_{se\_Req}$; "Valid"; "Increase Dia") = Valid
Determine Emended Length

Factor (According to Cl.D.5.2.6 of ACI318), \( \psi_{c,N} = 1.00 \)

Effective Embedment Length (According to Cl.D.5.2.1 of ACI318),

\[
\begin{align*}
\text{h}_{\text{ef,Req}} &= \left( \frac{P_u}{\Phi_2 \cdot \psi_{c,N} \cdot 24 \cdot \lambda \cdot \sqrt{f'_c}} \right)^{2/3} = 3.51 \text{ in} \\
\text{Provided Embedment Length, h}_{\text{ef,Prov}} &= 4.00 \text{ in} \\
\text{Check Validity} &= \text{IF} (\text{h}_{\text{ef,Prov}} \geq \text{h}_{\text{ef,Req}}; "\text{Valid}"; "\text{Increase h}_{\text{ef}}") = \text{Valid}
\end{align*}
\]

Determine Head Size

Factor (According to Cl.D.5.3.6 of ACI318), \( \psi_{c,P} = 1.00 \)

Required Head Size for Anchor Bolt (According to Eq.D-15 of ACI318),

\[
A_{\text{brg}} = \frac{P_u}{\Phi_2 \cdot \psi_{c,P} \cdot 8 \cdot f'_c} = 0.313 \text{ in}^2
\]

Design Summary

Diameter of Anchor Bolt, Dia = 0.625 in
Embedment Length of Anchor Bolt, \( h_{\text{ef}} = h_{\text{ef,Prov}} \) = 4.00 in
Head Size of Anchor Bolt, \( A_{\text{brg}} = A_{\text{brg}} \) = 0.313 in\(^2\)
Design a Single Headed Anchor Bolt in Shear Near an Edge as per ACI 318-11 Appendix D

System

Edge Distance, e = 1.75 in

Load

Ultimate Load, \( V_u = 700 \text{ lb} \)

Material Properties

Concrete Strength, \( f'_c = 4000 \text{ psi} \)

Tensile Strength of Anchor Bolt Grade, \( f_{uta} = 58000 \text{ psi} \)

Strength Reduction Factor (According to Cl.D.4.4.a of ACI318), \( \phi_1 = 0.65 \)

Strength Reduction Factor (According to Cl.D.4.4.c of ACI318), \( \phi_2 = 0.70 \)

Modification Factor for Lightweight Concrete, \( \lambda = 1.00 \)

Determine Anchor Diameter

Required Effective Area of Anchor Bolt (According to Eq.D.29 of ACI318),

\[
A_{se\_Req} = \frac{V_u}{\phi_1 \times 1.0 \times 0.6 \times f_{uta}} = 0.031 \text{ in}^2
\]

Provided Anchor Bolt, Dia = SEL("ACI/Anchor"; Dia; ) = 0.500 in

Provided Area of Anchor Bolt, \( A_{se\_Prov} = \text{TAB("ACI/Anchor"; Ase; Dia=Dia)} = 0.142 \text{ in}^2 \)

Check Validity = IF(\( A_{se\_Prov} \geq A_{se\_Req} ";" \text{"Valid"}; "Increase Dia") = Valid
Chapter 1: Concrete Design

Single Headed Anchor Bolt in Shear Near an Edge

Calculation of Embedment Strength

Assume that, \( h_{ef,Prov} = 7.00 \text{ in} \)

Ratio \( A_vc/A_{vc0}, A' = 1.00 \)

Factor (According to Cl.D.6 of ACI318), \( \psi_{ed,V} = 1.00 \)

Factor (According to Cl.D.6.2.7 of ACI318), \( \psi_{c,V} = 1.00 \)

Length of Load Bearing of Anchor Bolt (According to Cl.D.6.2.2 of ACI318), \( l_e = \text{MIN}(h_{ef,Prov}; 8\times\text{Dia}) = 4.00 \text{ in} \)

Basic Strength of Concrete Breakout (According to Eq.D-33 of ACI318),

\[
V_{b1} = 7 \times \left( \frac{l_e}{\text{Dia}} \right)^{0.2} \times \sqrt{\text{Dia} \times \lambda \times \sqrt{f'c} \times \varepsilon^{1.5}} = 1098 \text{ lb}
\]

Basic Strength of Concrete Breakout (According to Eq.D-34 of ACI318),

\[
V_{b2} = 9 \times \lambda \times \sqrt{f'c} \times \varepsilon^{1.5} = 1318 \text{ lb}
\]

Basic Strength of Concrete Breakout, \( V_b = \text{MIN}(V_{b1} ; V_{b2}) = 1098 \text{ lb} \)

Nominal Strength of Concrete Breakout (According to Eq.D-30 of ACI318),

\[
\Phi V_{cb} = A' \times \Phi_2 \times \psi_{ed,V} \times \psi_{c,V} \times V_b = 769 \text{ lb}
\]

Check Validation = IF \( V_u \leq \Phi V_{cb}; "Valid"; "Invalid" \) = Valid

Design Summary

Diameter of Anchor Bolt, \( \text{Dia} = 0.500 \text{ in} \)

Embedment Length of Anchor Bolt, \( h_{ef} = h_{ef,Prov} = 7.00 \text{ in} \)
Calculation of Deflection for Simple Support Concrete Beam Under Uniform Loads

As per ACI 318-11 Chapter 9

System

Width of Concrete Section, b = 12.0 in
Depth of Concrete Section, h = 22.0 in
Concrete Cover, co = 2.5 in
Effective Depth of Concrete Section, d = h - co = 22.0 - 2.5 = 19.5 in
Depth of Compression Reinforcement, d' = 2.5 in
Area of Tension Reinforcement, A_s = 1.80 in^2
Area of Compression Reinforcement, A_s' = 0.6 in^2
ρ = \frac{A_s}{(b \times d)} = 0.0077
ρ' = \frac{A_s'}{(b \times d)} = 0.0026
Distance from Centroidal Axis of Gross Section, y_t = h / 2 = 11.0 in
Beam Span, L = 25.0 ft

Load

Uniform Dead Load, w_D = 0.395 kip/ft
Uniform Live Load, w_L = 0.300 kip/ft
Percentage of Sustained Live Load, Sus = 50 %

Moment due to Dead Load, M_D = w_D \times \frac{L^2}{8} = 30.9 kip*ft
Moment due to Live Load, M_L = w_L \times \frac{L^2}{8} = 23.4 kip*ft
Sustained Moment, M_{sus} = M_D + (Sus/100)\times M_L = 42.6 kip*ft

Material Properties

Concrete Strength, f'_c = 3000 psi
Yield Strength of Reinforcement, f_y = 40000 psi
Modulus of Elasticity of Reinforcement, E_s = 29000000 psi
Modification Factor for Lightweight Concrete, \lambda = 1.00
Concrete Density, w_c = 150 psi
Properties of Cracked Section

Modulus of Rupture (According to Eq. 9-10 of ACI318), \( f_r = 7.5 \cdot \lambda \cdot \sqrt{f_c} \) = 411 psi

Modulus of Elasticity of Concrete (According to Cl. 8.5.1 of ACI318),

\[
E_c = w_c^{1.5} \cdot 33 \cdot \sqrt{f_c} = 3320561 \text{ psi}
\]

\[
n_s = \frac{E_s}{E_c} = 8.7
\]

\[
l_g = \frac{b \cdot h^3}{12} = 10648 \text{ in}^4
\]

\[
B = \frac{b}{(n_s \cdot A_s)} = 0.77 \text{ in}
\]

\[
r = \frac{(n_s - 1) \cdot A_s'}{n_s \cdot A_s} = 0.295
\]

\[
kd = \frac{\sqrt{2 \cdot d \cdot B \left(1 + \frac{d'}{d} \right) + \left(1 + r \right)^2 - (1 + r)}}{B} = 5.76 \text{ in}
\]

\[
l_{cr} = \frac{b \cdot kd^3}{3} + n_s \cdot A_s \cdot (d - kd) + (n_s - 1) \cdot A_s' \cdot (kd - d')^2 = 3770 \text{ in}^4
\]

Cracking Moment (According to Eq. 9-9 of ACI318),

\[
M_{cr} = f_r \cdot l_g \left( \gamma_t \cdot 12000 \right) = 33.2 \text{ kip} \cdot \text{ft}
\]

Properties of Effective Section

Effective Moment of Inertia (According to Eq 9-8 of ACI318):

\[
l_{e, Dead1} = \left( \frac{M_{cr}}{M_D} \right)^3 \cdot l_g + \left(1 - \left( \frac{M_{cr}}{M_D} \right)^3 \right) \cdot l_{cr} = 12301 \text{ in}^4
\]

\[
l_{e, Dead} = \text{MIN}(l_g; l_{e, Dead1}) = 10648 \text{ in}^4
\]

\[
l_{e, Sus1} = \left( \frac{M_{cr}}{M_{sus}} \right)^3 \cdot l_g + \left(1 - \left( \frac{M_{cr}}{M_{sus}} \right)^3 \right) \cdot l_{cr} = 7026 \text{ in}^4
\]

\[
l_{e, Sus} = \text{MIN}(l_g; l_{e, Sus1}) = 7026 \text{ in}^4
\]

\[
l_{e, All1} = \left( \frac{M_{cr}}{M_D + M_L} \right)^3 \cdot l_g + \left(1 - \left( \frac{M_{cr}}{M_D + M_L} \right)^3 \right) \cdot l_{cr} = 5342 \text{ in}^4
\]

\[
l_{e, All} = \text{MIN}(l_g; l_{e, All1}) = 5342 \text{ in}^4
\]
Chapter 1: Concrete Design
Deflection of Simple Beam

Short Term Deflection

\[ \Delta_{i_{\text{Dead}}} = \frac{5 \cdot M_D \cdot L^2 \cdot 12^3}{48 \cdot E_c \cdot I_{e_{\text{Dead}}} / 1000} = 0.098 \text{ in} \]

\[ \Delta_{i_{\text{Sus}}} = \frac{5 \cdot M_{\text{Sus}} \cdot L^2 \cdot 12^3}{48 \cdot E_c \cdot I_{e_{\text{Sus}}} / 1000} = 0.205 \text{ in} \]

\[ \Delta_{i_{\text{All}}} = \frac{5 \cdot (M_D + M_L) \cdot L^2 \cdot 12^3}{48 \cdot E_c \cdot I_{e_{\text{All}}} / 1000} = 0.344 \text{ in} \]

\[ \Delta_{i_{\text{Live}}} = \Delta_{i_{\text{All}}} - \Delta_{i_{\text{Dead}}} = 0.246 \text{ in} \]

Long Term Deflection

Duration of Sustained loads, Dur: SEL("ACI/Sustained";Dur; ) = 5 Years or more

Time-Dependent Factor for Sustained Loads (According to Cl. 9.5.2.5 of ACI318):

\[ \xi = \text{TAB("ACI/Sustained";x;Dur=Dur; )} = 2.00 \]

Multiplier Factor for Long-Term Deflection (According to Eq. 9-11 of ACI318),

\[ \lambda_{\Delta} = \xi / (1 + 50 \cdot \rho') = 1.77 \]

Creep and Shrinkage Deflection, \( \Delta_{cp_{-}\text{sh}} = \lambda_{\Delta} \cdot \Delta_{i_{\text{Sus}}} \)

\[ \Delta_{\text{total}} = \Delta_{cp_{-}\text{sh}} + \Delta_{i_{\text{Live}}} = 0.61 \text{ in} \]

Calculation Summary

Long Term Deflection, \( \Delta_{\text{total}} = \Delta_{cp_{-}\text{sh}} + \Delta_{i_{\text{Live}}} = 0.61 \text{ in} \)

Interactive Design Aids for Structural Engineers
Design of Shear Reinforcement for Section Subject to Shear & Flexure as per ACI318-11 Chapter 11

System

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of Concrete Section, b=</td>
<td>13.0 in</td>
</tr>
<tr>
<td>Depth of Concrete Section, h=</td>
<td>22.5 in</td>
</tr>
<tr>
<td>Concrete Cover, co=</td>
<td>2.5 in</td>
</tr>
<tr>
<td>Effective Depth of Concrete Section, d=</td>
<td>20.0 in</td>
</tr>
<tr>
<td>Beam Span, L=</td>
<td>30.0 ft</td>
</tr>
</tbody>
</table>

Load

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Uniform Load, w_u=</td>
<td>4.5 kip/ft</td>
</tr>
<tr>
<td>Ultimate Shear Force at Support, V_ui=</td>
<td>w_u*L/2 = 67.5 kips</td>
</tr>
<tr>
<td>Ultimate Shear Force at Distance [d] from Support, V_u= V_ui - w_u * (d/12)</td>
<td>60.0 kips</td>
</tr>
</tbody>
</table>

Material Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Strength, f'_c=</td>
<td>3000 psi</td>
</tr>
<tr>
<td>Yield Strength of Reinforcement, f_y=</td>
<td>40000 psi</td>
</tr>
<tr>
<td>Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), ( \Phi )=</td>
<td>0.75</td>
</tr>
<tr>
<td>Modification Factor for Lightweight Concrete, ( \lambda )=</td>
<td>1.00</td>
</tr>
<tr>
<td>Concrete Density, w_c=</td>
<td>150 psi</td>
</tr>
</tbody>
</table>

Determine Concrete Shear Strength

Nominal Shear Strength provided by Concrete (According to Eq. 11-3 of ACI318),

\[
V_c = 2 * \lambda * \sqrt{f'_c * b * d / 1000} = 28.5 \text{ kips}
\]

Shear Reinforcement is:

\[
\text{IF}(V_u > \Phi * V_c; "Required","Not Required") = \text{Required}
\]
Determine Area of Shear Reinforcement

Nominal Shear Strength provided by Reinforcement (According to Eq. 11-2 of ACI318),
\[ V_s = \frac{V_u - \phi \cdot V_c}{\phi} = 51.5 \text{ kips} \]

Maximum Allowable Shear Strength provided by Reinforcement (According to Cl.11.4.7.9 of ACI318),
\[ V_{s_{\text{max}}} = 8 \cdot \sqrt{f_c' \cdot b \cdot d \cdot 1000} = 113.9 \text{ kips} \]

IF\(V_s>V_{s_{\text{max}}}; \text{"Increase Beam Dimension"}; \text{"OK"} \) = OK

Spacing of Provided Stirrups, \(s=6.0 \text{ in} \)

Required Area of Reinforcement, \(A_v = \frac{V_s \cdot s \cdot 1000}{f_y \cdot d} = 0.39 \text{ in}^2 \)

Minimum Area of Reinforcement (According to Cl.11.4.6.3 of ACI318),
\[ A_{v_{\text{min}1}} = \frac{0.75 \cdot \sqrt{f_c' \cdot b \cdot s}}{f_y} = 0.08 \text{ in}^2 \]
\[ A_{v_{\text{min}2}} = \frac{50 \cdot b \cdot s}{f_y} = 0.10 \text{ in}^2 \]
\[ A_v = \text{MAX}(A_{v_{\text{min}1}}; A_{v_{\text{min}2}}) = 0.10 \text{ in}^2 \]

Required Area of Reinforcement, \(A_{v_{\text{Req}}} = \text{MAX}(A_v; A_{v_{\text{min}}}) = 0.39 \text{ in}^2 \)

Provided Reinforcement, \(\text{Bar}= \text{SEL}(\text{"ACI/Bar"}; \text{Bar};) = \text{No.4} \)

Provided Reinforcement, \(A_{sb} = \text{TAB}(\text{"ACI/Bar"}; \text{Asb}; \text{Bar}=\text{Bar}) = 0.20 \text{ in}^2 \)

Number of Stirrups, \(n=1 \)

Provided Area of Reinforcement, \(A_{v_{\text{Prov}}} = A_{sb} \cdot n \cdot 2 = 0.40 \text{ in}^2 \)

Check Validity= IF\(A_{v_{\text{Prov}}} \geq A_{v_{\text{Req}}}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \)

Determine Maximum Permissible Spacing of Stirrups

Allowable Shear Strength provided by Reinforcement for Spacing Limit (According to Cl.11.4.5.3 of ACI318),
\[ V_{s_{\text{limit}}} = 4 \cdot \lambda \cdot \sqrt{f_c' \cdot b \cdot d \cdot 1000} = 57.0 \text{ kips} \]

Factor for Maximum Spacing of Stirrups, Fac = IF\(V_s \leq V_{s_{\text{limit}}}; \text{1}; 0.5 \) = 1.0

Maximum Spacing of Stirrups (According to Cl.11.4.5.1 of ACI318),
\[ s_{\text{max}} = \text{MIN}(d/2;24) \cdot \text{Fac} = 10.00 \text{ in} \]

Check Validity= IF\(s \leq s_{\text{max}}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \)

Interactive Design Aids for Structural Engineers
Determine Distribution Distance of Shear Reinforcement

Distance from Support beyond which Minimum Shear Reinforcement is Required,

\[ x_c = \frac{V_{ui} - \Phi \cdot V_c}{w_u} = 10.3 \text{ ft} \]

Distance from Support beyond which Concrete can carry Shear Force,

\[ x_m = \frac{V_{ui} - \Phi \cdot V_c / 2}{w_u} = 12.6 \text{ ft} \]

Design Summary

Provided Area of Shear Reinforcement, \( A_{vc,Prov} = A_{vc,Prov} = 0.40 \text{ in}^2 \)

Distance from Support beyond which Minimum Shear Reinforcement, \( x_c = x_c = 10.3 \text{ ft} \)

Distance from Support beyond which Concrete can carry Shear Force, \( x_m = x_m = 12.6 \text{ ft} \)
Check Shear Reinforcement at Opening as per ACI 318-11 Chapter 11

System

- Width of Beam, \( b = 4.3 \) in
- Width of Top Flange, \( b_f = 48.0 \) in
- Height of Beam, \( h = 26.0 \) in
- Height of Compression Strut, \( h_c = 4.0 \) in
- Height of Tension Strut, \( h_t = 12.0 \) in

Load

- Ultimate Shear Force at Center of Opening, \( V_u = 7.2 \) kips
- Ultimate Axial Tension Force in Tension Strut, \( T_u = -10.8 \) kips
- Ultimate Axial Compression Force in Compression Strut, \( N_u = 60.0 \) kips
- Ultimate Shear Force in Tension Strut, \( V_{ut} = 6.0 \) kips
- Ultimate Shear Force in Compression Strut, \( V_{uc} = 5.4 \) kips

Material Properties

- Concrete Strength for Beam, \( f'_{c1} = 6000 \) psi
- Concrete Strength for Topping, \( f'_{c2} = 3000 \) psi
- Yield Strength of Reinforcement, \( f_y = 60000 \) psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = 0.75 \)
- Modification Factor for Lightweight Concrete, \( \lambda = 1.00 \)
Calculation of Required Shear Reinforcement

Required Shear RFT, $A_{v_{Req}} = \frac{V_u}{\phi * f_y / 1000} = 0.16 \text{ in}^2$

Identification of bar = SEL("ACI/Bar" ;Bar; ) = No.3

Provided Reinforcement, $A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar)} = 0.11 \text{ in}^2$

Number of Bars, n = 2

Provided Shear Reinforcement, $A_{v_{Prov}} = n * A_{sb} = 0.22 \text{ in}^2$

Check Validity = IF($A_{v_{Prov}} \geq A_{v_{Req}}$; "Valid"; "Invalid") = Valid

Calculation of Shear Reinforcement of Tensile Strut

Strut Depth, $d_t = 0.8 * h_t = 9.6 \text{ in}$

Concrete Shear Strength for Tensile Strut (According to Eq.11-8 of ACI318),

$$V_{ct} = 2 \left( 1 + \frac{T_u * 1000}{500 * b * h_t} \right) * \lambda * \sqrt{f'_{c1} * b * d_t} \times 1000 = 3.72 \text{ kips}$$

Shear Reinforcement for Tensile Strut = IF($V_{ut} \leq \phi * V_{ct}$; "Not Required"; "Required") = Required

Spacing between Stirrups (According to Cl.11.4.5.1 of ACI318),

$$s = 0.75 * h_t = 9.00 \text{ in}$$

Required RFT Area, $A_{v_{t_{Req}}} = \frac{(V_{ut} * \phi * V_{ct} ) * s}{\phi * f_y * d_t / 1000} = 0.07 \text{ in}^2$

Identification of Bar = SEL("ACI/Bar" ;Bar; ) = No.3

Provided Reinforcement, $A_{sb} = \text{TAB("ACI/Bar"; Asb; Bar=Bar)} = 0.11 \text{ in}^2$

Number of Bars, n = 1

Provided Shear Reinforcement, $A_{v_{t_{Prov}}} = n * A_{sb} = 0.11 \text{ in}^2$

Check Validity = IF($A_{v_{t_{Prov}}} \geq A_{v_{t_{Req}}}$; "Valid"; "Invalid") = Valid

Calculation of Shear Reinforcement of Compression Strut

Strut Depth, $d_c = 0.8 * h_c = 3.2 \text{ in}$

Concrete Shear Strength for Tensile Strut (According to Eq.11-8 of ACI318),

$$V_{cc} = 2 \left( 1 + \frac{N_u * 1000}{2000 * b_t * h_c} \right) * \lambda * \sqrt{f'_{c2} * b_t * d_c} \times 1000 = 19.46 \text{ kips}$$

Shear Reinforcement for Compression Strut = IF($V_{uc} \leq \phi * V_{cc}$; "Not Required"; "Required") = Not Required
Spacing between Stirrups (According to Cl.11.4.5.1 of ACI318),
\[ s = 0.75 \times h_c = 3.00 \text{ in} \]

Required RFT Area, \( A_{vc_{-}Req} = \frac{(V_{uc} - \phi \times V_{cc}) \times s}{\phi \times f_y \times d_c / 1000} = -0.19 \text{ in}^2 \)

Identification of, Bar= \( SEL("ACI/Bar"; Bar;) = No.3 \)

Provided Reinforcement, \( A_{sb} = TAB("ACI/Bar"; Asb; Bar=Bar) = 0.11 \text{ in}^2 \)

Number of Bars, \( n = 1 \)

Provided Shear Reinforcement, \( A_{vc_{-}Prov} = n \times A_{sb} = 0.11 \text{ in}^2 \)

Check Validity= \( IF(A_{vc_{-}Prov} \geq A_{vc_{-}Req}; "Valid"; "Invalid") = Valid \)

**Design Summary**

Shear Reinforcement for Beam, \( A_{v_{-}Prov} = A_{v_{-}Prov} = 0.22 \text{ in} \)

Shear Reinforcement for Ten. Strut, \( A_{vt_{-}Prov} = IF(V_{ut} \leq \phi \times V_{ct}; "Zero"; A_{vt_{-}Prov}) = 0.11 \text{ in}^2 \)

Shear Reinforcement for Com. Strut, \( A_{vc_{-}Prov} = IF(V_{uc} \leq \phi \times V_{cc}; "Zero"; A_{vc_{-}Prov}) = Zero \)
Design of Horizontal Shear for Composite Slab and Precast Beam as per ACI 318-11 Chapters 11 & 17

System

- Width of Beam, \( b = \) 10.0 in
- Height of Beam, \( h = \) 20.5 in
- Concrete Cover, \( c_o = \) 1.5 in
- Depth of Beam, \( d = h - c_o = \) 19.0 in
- Span of Simple Beam, \( L = \) 30.0 ft
- Identification of, Bar = SEL("ACI/Bar" ;Bar; ) = No.5
- Diameter of Bars, \( d_b = \) TAB("ACI/Bar" ;Dia ;Bar=Bar ) = 0.63 in
- Number of Bars, \( n = \) 2

Load

- Service Dead Load, \( W_D = \) 315 lb/ft
- Service Live Load, \( W_L = \) 3370 lb/ft
- Ultimate Load, \( W_u = 1.2 \times W_D + 1.6 \times W_L = \) 5770 lb/ft

Material Properties

- Concrete Strength, \( f'_c = \) 3000 psi
- Yield Strength of Reinforcement, \( f_y = \) 60000 psi
- Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \phi = \) 0.75
- Modification Factor for Lightweight Concrete, \( \lambda = \) 1.00
- Friction Factor (According to Cl.11.6.4.3 of ACI318), \( \mu = 1.0 \times \lambda = \) 1.00
Chapter 1: Concrete Design

Horizontal Shear for Composite Slab and Precast Beam

Calculation of Horizontal Shear Reinforcement

Ultimate Shear Force at Distance (d) from Support

\[ V_u = \frac{\left( W_u \frac{L}{2} \right) - \left( W_u \frac{d}{12} \right)}{1000} = 77.4 \text{ kips} \]

Horizontal Shear Strength (According to Cl.17.5.3 of ACI318),

\[ \Phi V_{nh} = \frac{\Phi \cdot 500 \cdot b \cdot d}{1000} = 71.3 \text{ kips} \]

Horizontal Shear Reinforcement = IF \( V_u \leq \Phi V_{nh} ; "Not Required" ; "Required" \) = Required

Horizontal Shear Force Pre one foot, \( \nu_{uh} = \frac{V_u}{d \cdot b} = 0.407 \text{ ksi} \)

Required RFT Area for Shear Friction, \( A_{vf} = \frac{V_{uh} \cdot b \cdot 12}{\Phi \cdot f_y \cdot \mu / 1000} = 1.09 \text{ in}^2 / \text{ft} \)

Spacing Between Links, \( s = \frac{\pi \cdot n \cdot 12 \cdot d_b^2}{A_{vf} \cdot 4} = 6.9 \text{ in} \)

Design Summary

Spacing Between Links, \( s = 6.9 \text{ in} \)
Design of Reinforcement for Shallow Foundation as per ACI 318-11 Chapter 15

System

Width of Column, $c_1 =$ 30.0 in
Length of Column, $c_2 =$ 12.0 in
Width of Footing, $B =$ 13 ft
Length of Footing, $L =$ 13 ft
Area of Footing, $A_f =$ $B \times L$ = 169 ft$^2$
Depth of Footing, $t =$ 30.5 in
Concrete Cover, $c_0 =$ 2.5 in
Effective Depth of Footing, $d =$ $t - c_0$ = 28.0 in

Load

Service Dead Load, $P_D =$ 350 kips
Service Live Load, $P_L =$ 275 kips
Ultimate Load, $P_u =$ $1.2 \times P_D + 1.6 \times P_L$ = 860 kips
Ultimate Pressure, $q_s =$ $P_u / A_f$ = 5.09 ksf
Material Properties

Concrete Strength, \( f'_c = 3000 \text{ psi} \)

Yield Strength of Reinforcement, \( f_y = 60000 \text{ psi} \)

Tension Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi = 0.90 \)

Determine Area of Reinforcement Distributed in Footing Width (B)

\[
M_{u1} = q_s \cdot B \cdot \left( 0.5 \cdot \left( \frac{L - \frac{c_2}{12}}{c_2} \right) \right)^2 / 2 = 1191 \text{ kip*ft}
\]

\[
R_{n1} = \frac{M_{u1} \cdot 12000}{\Phi \cdot B \cdot 12 \cdot d^2} = 130 \text{ psi}
\]

\[
\rho_1 = \frac{0.85 \cdot f'_c \left( 1 - \sqrt{1 - \frac{2 \cdot R_{n1}}{0.85 \cdot f'_c}} \right)}{f_y} = 0.0022
\]

Minimum Reinforcement Ratio (According to Cl.7.12.2 of ACI318),

\[
\rho_{\text{min}} = \text{IF}(f_y \leq 50000; 0.002; \text{IF}(f_y > 77143; 0.0014; 0.0018)) = 0.0018
\]

Required Area of Reinforcement, \( A_{s1_{\text{Req}}} = \text{MAX}(\rho_{\text{min}}; \rho_1) \cdot B \cdot d \cdot 12 = 9.61 \text{ in}^2 \)

Provided Reinforcement, Bar = \( \text{SEL}("ACI/Bar"; \text{Bar}; ) = \text{No.8} \)

Provided Reinforcement, \( A_{s1_{\text{Prov}}} = \text{TAB}("ACI/Bar"; \text{Asb}; \text{Bar}=\text{Bar}) = 0.79 \text{ in}^2 \)

Number of Bars, \( n_1 = 13 \)

Provided Area of Reinforcement, \( A_{s1_{\text{Prov}}} = n_1 \cdot A_{s1_{\text{Prov}}} = 10.27 \text{ in}^2 \)

Check Validity = \( \text{IF}(A_{s1_{\text{Prov}}} \geq A_{s1_{\text{Req}}}; "Valid"; "Invalid") = \text{Valid} \)
Determine Area of Reinforcement Distributed in Footing Length (L)

\[ M_{u2} = q_s \times L \left( 0.5 \times \left( B - \frac{c_1}{12} \right) \right)^2 / 2 = 912 \text{ kip*ft} \]

\[ R_{n2} = \frac{M_{u2} \times 12000}{\phi \times L \times d^2} = 99 \text{ psi} \]

\[ \rho_2 = \frac{0.85 \times f_c \times \left( 1 - \sqrt{1 - \frac{2 \times R_{n2}}{0.85 \times f_c}} \right)}{f_y} = 0.0017 \]

Minimum Reinforcement Ratio (According to Cl.7.12.2 of ACI318),

\[ \rho_{\text{min}} = \text{IF}(f_y \leq 50000; 0.002; \text{IF}(f_y \geq 77143; 0.0014; 0.0018)) = 0.0018 \]

Required Area of Reinforcement, \( A_{s2_{\text{req}}} = \text{MAX}(\rho_{\text{min}}; \rho_2) \times L \times d \times 12 = 7.86 \text{ in}^2 \)

Provided Reinforcement, Bar= SEL("ACI/Bar"; Bar; ) = No.8

Provided Reinforcement, \( A_{sb2} = \text{TAB("ACI/Bar"; Asb; Bar=Bar)} = 0.79 \text{ in}^2 \)

Number of Bars, \( n_2 = 11 \)

Provided Area of Reinforcement, \( A_{s2_{\text{prov}}} = n_2 \times A_{sb2} = 8.69 \text{ in}^2 \)

Check Validity= \( \text{IF}(A_{s2_{\text{prov}}}) \geq A_{s2_{\text{req}}}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \)

**Design Summary**

Area of Reinforcement Distributed in Footing Width, \( A_{s1} = A_{s1_{\text{prov}}} = 10.27 \text{ in}^2 \)

Area of Reinforcement Distributed in Footing Length, \( A_{s2} = A_{s2_{\text{prov}}} = 8.69 \text{ in}^2 \)
Design for Depth of Shallow Foundation as per ACI 318-11 Chapters 11 & 15

System

Width of Column, \( c_1 \) = 30.0 in
Length of Column, \( c_2 \) = 12.0 in
Concrete Cover, \( c_0 \) = 5.0 in
Height of Soil above Footing, \( h_s \) = 5 ft

Load

Service Dead Load, \( P_D \) = 350 kips
Service Live Load, \( P_L \) = 275 kips
Ultimate Load, \( P_u \) = 1.2\( P_D \) + 1.6\( P_L \) = 860 kips
Service Surcharge, \( q \) = 0.1 ksf
Allowable Soil Pressure at Bottom of Footing, \( P_a \) = 4.5 ksf
Average Weight of Soil and Concrete above Footing Base, \( w \) = 130.0 pcf

Material Properties

Concrete Strength, \( f'_c \) = 3000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), \( \Phi \) = 0.75
Modification Factor for Lightweight Concrete, \( \lambda \) = 1.00
Chapter 2: Foundation Design

Depth of Shallow Foundation

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Calculation of Base Area

Net Allowable Soil Pressure, \( P_{na} = P_a - q \frac{w \cdot h_s}{1000} \) = 3.75 ksf

Required Area of Footing, \( A_f = \frac{P_D + P_L}{P_{na}} \) = 166.7 ft²

Assume Width of Footing, \( B = 13 \) ft
Assume Length of Footing, \( L = 13 \) ft
Check Validity = IF(\( A_f > L \cdot B \); "Invalid"; "Valid") = Valid

Ultimate Pressure, \( q_s = \frac{P_u}{(B \cdot L)} \) = 5.09 ksf

Calculation of Required Thickness for One-Way Action

Assume that Thickness of Footing, \( t = 33 \) in
Depth of Footing, \( d = t - c_0 \) = 28 in

Critical Area of One-Way Shear, \( A_{1B} = B \left( \frac{L - c_2 / 12}{2} \cdot \frac{d}{12} \right) \) = 47.67 ft²

Critical Area of One-Way Shear, \( A_{1L} = L \left( \frac{B - c_1 / 12}{2} \cdot \frac{d}{12} \right) \) = 37.92 ft²

Critical Area of One-Way Shear, \( A_1 = \text{MAX}(A_{1B}; A_{1L}) \) = 47.67 ft²

Width of Critical Section for One-Way Shear, \( b_w = \text{IF}(A_{1B} > A_{1L}; B; L) \) = 13 ft

Ultimate Shear force at Critical Area Section, \( V_{u1} = q_s \cdot A_1 \) = 243 kips

Nominal Concrete Shear Strength, \( \Phi V_c = \Phi \cdot 2 \cdot \lambda \cdot \sqrt{f_c' \cdot b_w \cdot 12 \cdot d} \) = 359 kips

Check Validation = IF(\( \Phi V_c > V_{u1} \); "O.K."; "Increase Depth") = O.K.

Calculation of Required Thickness for Two-Way Action

Critical Area of Two-Way Shear, \( A_2 = B \cdot L \left( \frac{(c_1 + d) \cdot (c_2 + d)}{144} \right) \) = 152.89 ft²

Ultimate Shear force at Critical Area Section, \( V_{u2} = q_s \cdot A_2 \) = 778.2 kips

Perimeter of Critical Section for Two-Way Shear, \( b_0 = 2(c_1 + d) + 2(c_2 + d) \) = 196.0 in

Column Type = SEL("ACI/Alfa S"; Type; ) = Interior

Alfa Constant, \( \alpha_s = \) TAB("ACI/AlfaS"; Alfa; Type=Type) = 40.00

Ratio of Long to Short Column Dimensions, \( \beta = \text{MAX}(c_1; c_2) / \text{MIN}(c_1; c_2) \) = 2.50

Interactive Design Aids for Structural Engineers
Concrete Shear Strength (According to Eq. 11-31 of ACI318),
\[ V_{c1} = \left( 2 + \frac{4}{\beta} \right) \lambda \sqrt{f'_c} \frac{b_0 \cdot d}{1000} = 1082 \text{ kips} \]

Concrete Shear Strength (According to Eq. 11-32 of ACI318),
\[ V_{c2} = \left( \frac{d}{b_0} + 2 \right) \alpha_s \sqrt{f'_c} \frac{b_0 \cdot d}{1000} = 2319 \text{ kips} \]

Concrete Shear Strength (According to Eq. 11-33 of ACI318),
\[ V_{c3} = 4 \lambda \sqrt{f'_c} \frac{b_0 \cdot d}{1000} = 1202 \text{ kips} \]

Nominal Concrete Shear Strength, \( \Phi V_c = \Phi \cdot \text{MIN}(V_{c1}; V_{c2}; V_{c3}) \) = 812 kips

Check Validation = IF( \( \Phi V_c > V_{u2} \); "O.K."; "Increase Depth" ) = O.K.

**Calculation Summary**
- Width of Footing, B = 13 ft
- Length of Footing, L = 13 ft
- Thickness of Footing, t = 33 in
Design Depth for Pile Cap as per ACI 318-11 Chapter 11

System

Width of Column, c₁ = 16.0 in
Length of Column, c₂ = 16.0 in
Pile Diameter, D = 12.0 in
Edge Distance for Corner Pile, e₁ = 15 in
Edge Distance for Corner Pile, e₂ = 15 in
Width of Pile Cap, B = 8.5 ft
Length of Pile Cap, L = 8.5 ft
Concrete Cover, c₀ = 7.0 in

Load

Pile Service Dead Load, P₇ = 20 kips
Pile Service Live Load, P₈ = 10 kips
Ultimate Pile Load, Pₚ = 1.2 * P₇ + 1.6 * P₈ = 40 kips

Material Properties

Concrete Strength, f'ₙ = 4000 psi
Shear Strength Reduction Factor (According to Cl.9.3.2 of ACI318), Φ = 0.75
Modification Factor for Lightweight Concrete, λ = 1.00
Chapter 2: Foundation Design

Depth for Pile Cap

Calculation of Required Thickness due to One-Way Shear

Assume that Thickness of Pile Cap, \( t = 22 \) in
Depth of Pile Cap, \( d = t - co = 15 \) in
Width of Critical Section for One-Way Shear, \( b_w = \text{MIN}(B ; L) = 8.5 \) ft
Number of Piles fall within Critical Section for One-Way Action, \( n_{r1} = 3 \)
Ultimate Shear force at Critical Area Section, \( V_{u1} = P_u * n_{r1} = 120 \) kips
Nominal Concrete Shear Strength, \( \Phi V_c = \Phi * 2 * \lambda * \sqrt{f'_c * b_w * 12*d} / 1000 \) = 145 kips
Check Validation = IF( \( \Phi V_c > V_{u1}; \) "O.K.;" "Increase Depth" ) = O.K.

Calculation of Required Thickness due to Two-Way Shear for Group Piles

Perimeter of Critical Section for Two-Way Shear, \( b_0 = 2*(c_1+d) + 2*(c_2+d) = 124.0 \) in
Number of Piles fall within Critical Section for Two-Way Action, \( n_{r2} = 8 \)
Ultimate Shear force at Critical Area Section, \( V_{u2} = P_u * n_{r2} = 320 \) kips
Column Type = SEL("ACI/Alfa S"; Type; ) = Interior
Alfa Constant, \( \alpha_s = \) TAB("ACI/AlfaS"; Alfa; Type=Type) = 40.00
Ratio of Long to Short Column Dimensions, \( \beta = \text{MAX}(c_1;c_2)/\text{MIN}(c_1;c_2) = 1.00 \)
Concrete Shear Strength (According to Eq. 11-31 of ACI318),
\[ V_{c1} = \left( 2 + \frac{4}{\beta} \right) * \lambda * \sqrt{f'_c * b_0 * d} / 1000 \] = 706 kips
Concrete Shear Strength (According to Eq. 11-32 of ACI318),
\[ V_{c2} = \left( \alpha_s * \frac{d}{b_0} + 2 \right) * \lambda * \sqrt{f'_c * b_0 * d} / 1000 \] = 804 kips
Concrete Shear Strength (According to Eq. 11-33 of ACI318),
\[ V_{c3} = 4 * \lambda * \sqrt{f'_c * b_0 * d} / 1000 \] = 471 kips
Nominal Concrete Shear Strength, \( \Phi V_c = \Phi * \text{MIN}(V_{c1}; V_{c2}; V_{c3}) = 353 \) kips
Check Validation = IF( \( \Phi V_c > V_{u2}; \) "O.K.;" "Increase Depth" ) = O.K.
Calculation of Required Thickness due to Two-Way Shear for Single Corner Pile

Perimeter of Critical Section for Two-Way Shear, \( b_{01} = \pi \times (D + d) \) = 84.8 in

Perimeter of Critical Section for Two-Way Shear, \( b_{02} = \pi \times (D+d)/4 + e_{s1} + e_{s2} \) = 51.2 in

Perimeter of Critical Section for Two-Way Shear, \( b_0 = \text{MIN}(b_{01}; b_{02}) \) = 51.2 in

Perimeter Ultimate Shear force at Critical Section, \( V_{u3} = P_u \times 1.0 \) = 40 kips

Column Type = SEL("ACI/Alfa S"; Type; ) = Corner

Alfa Constant, \( \alpha_s \) = TAB("ACI/AlfaS"; Alfa; Type=Type) = 20.00

Ratio of Long to Short Column Dimensions, \( \beta = \text{MAX}(c_1; c_2)/\text{MIN}(c_1; c_2) \) = 1.00

Concrete Shear Strength (According to Eq. 11-31 of ACI318), \( V_{c1} = \left(2 + \frac{4}{\beta}\right) \times \lambda_1 \times \sqrt{f'c} \times \frac{b_0 \times d}{1000} \) = 291 kips

Concrete Shear Strength (According to Eq. 11-32 of ACI318), \( V_{c2} = \left(\alpha_s \times \frac{d}{b_0} + 2\right) \times \lambda_1 \times \sqrt{f'c} \times \frac{b_0 \times d}{1000} \) = 382 kips

Concrete Shear Strength (According to Eq. 11-33 of ACI318), \( V_{c3} = 4 \times \lambda_1 \times \sqrt{f'c} \times \frac{b_0 \times d}{1000} \) = 194 kips

Nominal Concrete Shear Strength, \( \Phi V_c = \Phi \times \text{MIN}(V_{c1}; V_{c2}; V_{c3}) \) = 146 kips

Check Validation = IF( \( \Phi V_c > V_{u3} \); "O.K."; "Increase Depth" ) = O.K.

Calculation Summary

Thickness of Pile Cap, \( t = t \) = 22 in
Design of Slab on Grade Due to Wheel Load as per ACI 360-10 Appendix 1

System

- Thickness of Slab on Grade, \( h = 10.0 \text{ in} \)
- Spacing between Wheels, \( s = 40.0 \text{ in} \)
- Contact Area per Wheel, \( A_c = 50.0 \text{ in}^2 \)
- Effective Contact Area per Wheel (According to Cl.A1.2 of ACI360), \( A_{c\_eff} = 61.5 \text{ in}^2 \)

Load

- Wheel Axle Load, \( P = 30 \text{ kips} \)

Material Properties

- Concrete Strength, \( f_c' = 4000 \text{ psi} \)
- Subgrade Modulus, \( K = 100 \text{ lb/in}^3 \)
- Safety Factor, \( FS = 2.00 \)

Checking Slab Thickness

- Modulus of Rupture of Concrete, \( f_r = 9 \sqrt{f_c'} = 569.2 \text{ psi} \)
- Concrete Working Stress, \( f_{t\_all} = f_r / FS = 284.6 \text{ psi} \)
- Slab Stress per 1000 lb Axle Load, \( f_t = f_{t\_all} / (P/1.0) = 9.5 \text{ psi} \)

- Required Slab Thickness (According to Fig.A1.1 of ACI360), \( h_{min} = 9.92 \text{ in} \)
- Check Validation = IF\((h \geq h_{min}; \text{"Valid"}; \text{"Invalid"}) = \text{Valid} \)

Design Summary

- Thickness of Slab on Grade, \( h = 10.0 \text{ in} \)
Chapter 3: Steel Design

Design of W-Shapes Subjected to Bending Moment about Strong Axis and Braced at Some Points

Beam braced at some points

Materials

Grade: \( SEL("Material/ASTM"; NAME; ) = A992 \)

\( F_y = TAB("Material/ASTM";F_y;NAME=Grade) = 50 \text{ ksi} \)

\( E = 29000 \text{ ksi} \)

Beam Length and \( C_b \)

Total length, \( L = 35.00 \text{ ft} \)

Unsupported length, \( L_b = 17.50 \text{ ft} \)

From Table 3-1 (AISC ), \( C_b = 1.50 \)

Design Moments and Uniform Live Load

Ultimate moment, \( M_u = 200.00 \text{ kip*ft} \)

Ultimate moment due to live load case, \( M_L = 140.00 \text{ kip*ft} \)

Ultimate shear, \( Q_u = 30.00 \text{ kips} \)

Section Details

\( \text{sec.:} = W21X48 \)

\( d = 20.60 \text{ in} \)

\( t_w = 0.35 \text{ in} \)

\( b_f = 8.14 \text{ in} \)

\( t_f = 0.43 \text{ in} \)

\( Z_x = 107.00 \text{ in}^3 \)

\( S_x = 93.00 \text{ in}^3 \)

\( I_x = 959.00 \text{ in}^4 \)

\( r_y = 1.66 \text{ in} \)

\( r_{ts} = 2.05 \text{ in} \)

\( (r_y \text{ is radius of gyration about y-axis and } r_{ts} \text{ is effective radius of gyration for the L.T.B.}) \)
Chapter 3: Steel Design  
W-Shapes in Strong Axis Bending, Braced at Some Points

Torsional constant, \( J = \) TAB("AISC/W";J;NAME=sec.) = 0.80 in\(^4\)  
\( h_o = \) TAB("AISC/W";h\(_o\);NAME=sec.) = 20.20 in  

\((h_o : \text{is the distance between C.L. of flanges})\)

AISC Specification Eqn. (F2-1):  
Yielding Moment, \( M_p = \frac{Z_x F_y}{12} \) = 446 kip*ft

Element Classification

(1) Web:  
\( h/t_w, \lambda_w = \) TAB("AISC/W";h/t\(_w\);NAME=sec.) = 53.60  
According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:
Web._Class = IF(\( \lambda_w \leq 3.76 \sqrt{(E/F_y)} \),"Compact"; "Non-Compact") = Compact

(2) Comp. flange:  
\( b_f/2t_f, \lambda_f = \) TAB("AISC/W";bf/2tf;NAME=sec.) = 9.47  
According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:
\( \lambda_{pf} = 0.38 \sqrt{(E/F_y)} \) = 9  
\( \lambda_{rf} = 1.00 \sqrt{(E/F_y)} \) = 24

Fl._Class = IF(\( \lambda_f \leq \lambda_{pf} \),"Compact";IF(\( \lambda_f > \lambda_{rf} \),"Slender";"Non-Compact") ) = Non-Compact

The available strength provided by AISC Specification Sections F3.1 and F3.2, the nominal flexural moment is calculated as follows, satisfying the condition of compression Flange Local Buckling:
\( M_{n1a} = M_p - 0.7 F_y S_x \frac{1}{12} \) = 175 kip*ft  
\( M_{n1} = \text{IF(}\text{Fl._Class="Compact"; } M_p; \ (M_p - M_{n1a} \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}})) \) = 441 kip*ft

Lateral Torsional Buckling (LTB)

The limiting lengths \( L_p \) and \( L_r \) are determined according to the AISC Spec. Eqns. F2-5 and F2-6, as follows:
\( L_p = 1.76 r_y \sqrt{(E/F_y)/12} \) = 5.86 ft  
\( L_{r1} = \sqrt{\frac{J \times 1.0}{S_x \times h_o}} \) = 0.02  
\( L_{r2} = \sqrt{1 + \sqrt{\left(\frac{0.7 F_y S_x h_o}{E \times J \times 1.0}\right)^2}} \) = 2.89  
\( L_r = 1.95/12 r_{ts} \frac{E}{0.7 F_y} \times L_{r1} \times L_{r2} \) = 15.95 ft

Interactive Design Aids for Structural Engineers
Case = IF($L_b > L_r$; "ELTB"; IF($L_b \leq L_p$; "No LTB"; "InLTB")) = ELTB

("ELTB" refers to elastic LTB. and "InLTB" refers to inelastic LTB.)

According to the AISC Spec. Eqn. F2-2:

\[ M_1 = \min(M_p; C_b \cdot (M_p - 0.7112 F_y S_x) \cdot (L_b - L_p) / (L_r - L_p)) = 367 \text{kip*ft} \]

According to the AISC Spec. Eqn. F2-4:

\[ F_{cr} = C_b \cdot \pi^2 E / ((L_b + 0.0112)12/r_{ts})^2 = 40.87 \text{ksi} \]

\[ F_{cr,mod} = \sqrt{(1 + 0.078 \cdot J \cdot 1.0 / (S_x h_o) \cdot (L_b \cdot 12/r_{ts})^2} = 1.16 \text{ksi} \]

According to the AISC Spec. Eqns. F2-3:

\[ M_2 = \min(M_p, F_{cr,mod} S_x / 12 F_{cr,mod}) = 367 \text{kip*ft} \]

According to the AISC Spec. Eqn. F2-2:

\[ M_{n2} = \begin{cases} M_p & \text{Case = "No LTB";} \\ M_1 & \text{Case = "InLTB";} \\ M_2 & \text{Case = "ELTB";} \end{cases} = 367 \text{kip*ft} \]

Check The Available Flexure Strength

\[ \Phi M_n = 0.90 \min(M_p; M_{n1}; M_{n2}) = 330 \text{kip*ft} \]

Safety = IF($\Phi M_n \geq M_u$; "Safe"; "Unsafe") = Safe

Moment_ratio = $M_u / \Phi M_n = 0.61$

Check Shear Strength

\[ h/w, \lambda_w = \text{TAB("AISC/W";h/w;NAME=sec.)} = 53.60 \]

\[ \lambda_{w0} = 2.24 \cdot \sqrt{(E/F_y)} = 54 \]

\[ \lambda_{w1} = 1.10 \cdot \sqrt{(5E/F_y)} = 59 \]

\[ \lambda_{w2} = 1.37 \cdot \sqrt{(5E/F_y)} = 74 \]

Except for very few sections, which are listed in the User Note, AISC Specification Section G2.1(a) is applicable to the I-shaped beams published in the AISC Manual for $F_y = 50$ ksi. $C_v$ is calculated exactly according to Eqns. G2-2, G2-3, G2-4, and G2-5

\[ C_{va} = 1.5115E / (F_y \cdot \lambda_w^2) = 1.52 \]

\[ C_v = \begin{cases} \Phi (\lambda_w < \lambda_{w0}) & \Phi (\lambda_w > \lambda_{w1} \text{ AND } \lambda_w < \lambda_{w2}) & \lambda_{w1} / \lambda_{w2} & C_{va} \end{cases} = 1.00 \]

From AISC Specification Section G2.1b,

\[ A_w = d \cdot t_w = 7 \text{in}^2 \]

From AISC Specification Section G2.1, the available shear strength is:

\[ V_n = 0.6 F_y A_w C_v = 210 \text{kips} \]

\[ \Phi_v = 1.00 \]

\[ \Phi_v V_n = \Phi_v V_n = 210 \text{kips} \]

Shear_safety = IF($\Phi_v V_n > Q_u$; "Safe"; "Unsafe") = Safe
Check Deflection

\[ \Delta_{all} = \frac{12 \cdot L}{360} = 1.17 \text{ in} \]

\[ W_{eq} (LL), W_L = \frac{8 \cdot M_L}{L^2} = 0.91 \text{ kip/ft} \]

\[ \Delta_{act} = \frac{5 \cdot W_L \cdot L^4}{12 \cdot 384 \cdot E \cdot I_x} = 1.10 \text{ in} \]

Deflection safety \((D_s)\):

\[ D_s = \text{IF}(\Delta_{all} \geq \Delta_{act}; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe} \]

Design Summary

\[ \phi M_n = 0.90 \cdot \text{MIN}(M_p;M_{n1};M_{n2}) = 330 \text{ kip*ft} \]

Safety = \( \text{IF}(\phi M_n \geq M_u; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe} \)

Moment\_ratio = \( \frac{M_u}{\phi M_n} = 0.61 \)

\[ \phi V_n = \phi v \cdot V_n = 210 \text{ kips} \]

Shear\_safety = \( \text{IF}(\phi v \cdot V_n > Q_u; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe} \)

\[ \Delta_{all} = \frac{12 \cdot L}{360} = 1.17 \text{ in} \]

\[ \Delta_{act} = \frac{5 \cdot W_L \cdot L^4}{384 \cdot E \cdot I_x} = 0.01 \text{ in} \]

\[ D_s = \text{IF}(\Delta_{all} \geq \Delta_{act}; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe} \]
Design of W-Shapes Subjected to Moment about Strong Axis and Continuously Braced

Materials

Grade: SEL("Material/ASTM";NAME; ) = A992

\[ F_y = \text{TAB("Material/ASTM";F_y;NAME=Grade)} = 50 \text{ ksi} \]

\[ E = \text{TAB("Material/ASTM";E;NAME=Grade)} = 29000 \text{ ksi} \]

Beam Length and Cb

Total length, L = 35.00 ft

Design Moments and Uniform Live Load

Ultimate moment, \( M_u = 200.00 \text{ kip*ft} \)

Ultimate moment due to live load case, \( M_L = 140.00 \text{ kip*ft} \)

Ultimate shear, \( Q_u = 30.00 \text{ kips} \)

Section Details

sec.: SEL("AISC/W";NAME; ) = W21X48

depth, \( d = \text{TAB("AISC/W";d;NAME=sec.)} = 20.60 \text{ in} \)

Web th., \( t_w = \text{TAB("AISC/W";t_w;NAME=sec.)} = 0.35 \text{ in} \)

Flange width, \( b_f = \text{TAB("AISC/W";b_f;NAME=sec.)} = 8.14 \text{ in} \)

Flange th., \( t_f = \text{TAB("AISC/W";t_f;NAME=sec.)} = 0.43 \text{ in} \)

Plastic sec. modulus, \( Z_x = \text{TAB("AISC/W";Z_x;NAME=sec.)} = 107.00 \text{ in}^3 \)

Elastic sec. modulus, \( S_x = \text{TAB("AISC/W";S_x;NAME=sec.)} = 93.00 \text{ in}^3 \)

Inertia about x-axis, \( I_x = \text{TAB("AISC/W";I_x;NAME=sec.)} = 959.00 \text{ in}^4 \)

\[ r_y = \text{TAB("AISC/W";r_y;NAME=sec.)} = 1.66 \text{ in} \]

\[ r_{ts} = \text{TAB("AISC/W";r_{ts};NAME=sec.)} = 2.05 \text{ in} \]

\( (r_y \text{ is radius of gyration about y-axis and } r_{ts} : \text{is effective radius of gyration for the L.T.B.}) \)

Torsional constant, \( J = \text{TAB("AISC/W";J;NAME=sec.)} = 0.80 \text{ in}^4 \)

\[ h_o = \text{TAB("AISC/W";h_o;NAME=sec.)} = 20.20 \text{ in} \]

\( (h_o \text{ is the distance between C.L. of flanges}) \)

AISC Specification Eqn. (F2-1):

Yielding Moment, \( M_p = Z_x * F_y * 1/12 = 446 \text{ kip*ft} \)
Element Classification

(1) Web:
\[ h/t_w, \lambda_w = \text{TAB}("AISC/W";h/t_w;NAME=sec.) = 53.60 \]  According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:
\[ \text{Web Class} = \text{IF}(\lambda_w \leq 3.76 \cdot \sqrt{(E/F_y)}; "Compact"; "Non-Compact") = \text{Compact} \]

(2) Comp. flange:
\[ b_f/2t_f, \lambda_f = \text{TAB}("AISC/W";b_f/2t_f;NAME=sec.) = 9.47 \]  According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:
\[ \lambda_{pf} = 0.38 \cdot \sqrt{(E/F_y)} = 9 \]
\[ \lambda_{rf} = 1.00 \cdot \sqrt{(E/F_y)} = 24 \]  Flange Class:
\[ \text{IF}(\lambda_f \leq \lambda_{pf}, "Compact"; IF(\lambda_f > \lambda_{rf}, "Slender"; "Non-Compact")) = \text{Non-Compact} \]

Because the beam is continuously braced, and therefore not subjected to lateral-torsional buckling, the available strength is governed by AISC Specification Sections F3.1 and F3.2. The nominal flexural moment is calculated as follows, satisfying the condition of compression Flange Local Buckling:
\[ M_{n1a} = M_p - 0.7 \cdot F_y \cdot S_x \cdot 1/12 = 174.75 \text{ kip*ft} \]
\[ M_{n1} = \text{IF(Fl Class="Compact"; M_p; (M_p-(M_{n1a} \cdot (\lambda_f - \lambda_{pf})/\lambda_{rf} - \lambda_{pf}))) = 441 \text{ kip*ft} \]

Check The Available Flexure Strength
\[ \phi M_n = 0.90 \cdot \text{MIN}(M_p; M_{n1}) = 397 \text{ kip*ft} \]
Safety:
\[ \text{IF}(\phi M_n \geq M_u; "Safe"; "Unsafe") = \text{Safe} \]
Moment_ratio:
\[ M_u/\phi M_n = 0.50 \]

Check Shear Strength
\[ h/t_w, \lambda_w = \text{TAB}("AISC/W";h/tw;NAME=sec.) = 53.6 \]
\[ \lambda_{w0} = 2.24 \cdot \sqrt{(E/F_y)} = 53.9 \]
\[ \lambda_{w1} = 1.10 \cdot \sqrt{(5 \cdot E/F_y)} = 59.2 \]
\[ \lambda_{w2} = 1.37 \cdot \sqrt{(5 \cdot E/F_y)} = 73.8 \]

Except for very few sections, which are listed in the User Note, AISC Specification Section G2.1(a) is applicable to the I-shaped beams published in the AISC Manual for \( F_y = 50 \text{ ksi} \). \( C_v \) is calculated exactly according to Eqns. G2-2, G2-3, G2-4, and G2-5
\[ C_v = \text{IF}(\lambda_w \leq \lambda_{w0}; 1; \text{IF}(\lambda_w > \lambda_{w1} \text{ AND } \lambda_w \leq \lambda_{w2}; \lambda_{w1}/\lambda_w; 1.51 \cdot 5 \cdot E/(F_y \cdot \lambda_w^2)) = 1.00 \]
Chapter 3: Steel Design
W-Shape in Strong Axis Bending, Continuously Braced

From AISC Specification Section G2.1b,

\[ A_w = d \cdot t_w = 7 \text{ in}^2 \]

From AISC Specification Section G2.1, the available shear strength is:

\[ V_n = 0.6 \cdot F_y \cdot A_w \cdot C_v = 210 \text{ kips} \]

\[ \Phi_{v} = 1.00 \]

\[ \Phi_{v} \cdot V_n = \Phi_{v} \cdot V_n = 210 \text{ kips} \]

Shear_safety = IF(\( \Phi_{v} \cdot V_n > Q_u \); "Safe"; "Unsafe") = Safe

Check Deflection

\[ \Delta_{all} = \frac{12 \cdot L}{360} = 1.17 \text{ in} \]

\[ W_{eq (LL)}, W_L = \frac{8 \cdot M_L}{L} = 0.91 \text{ kip/ft} \]

\[ \Delta_{act} = \frac{12^3 \cdot 5 \cdot W_L \cdot L^4}{384 \cdot E \cdot I_x} = 1.10 \text{ in} \]

Deflection safety (D_s):

\[ D_s = \text{IF}(\Delta_{all} \geq \Delta_{act}; "Safe"; "Unsafe, increase section") = \text{Safe} \]

Design Summary

\[ \Phi M_n = 0.90 \cdot \text{MIN}(M_p; M_{n1}) = 397 \text{ kip*ft} \]

Safety = IF(\( \Phi M_n \geq M_u \); "Safe"; "Unsafe") = Safe

Moment_ratio = \( M_u / \Phi M_n \) = 0.50

\[ \Delta_{all} = \frac{12 \cdot L}{360} = 1.17 \text{ in} \]

\[ \Delta_{act} = \frac{5 \cdot W_L \cdot L^4}{384 \cdot E \cdot I_x} = 0.01 \text{ in} \]

\[ D_s = \text{IF}(\Delta_{all} \geq \Delta_{act}; "Safe"; "Unsafe, increase section") = \text{Safe} \]
Design of W-shapes Subjected to Moment about Minor Axis

Materials
Grade: SEL("Material/ASTM"; NAME; ) = A992
\( F_y \) = TAB("Material/ASTM";F\(_y\);NAME=Grade) = 50 ksi
E= 29000 ksi

Beam Length
Total length, \( L \) = 35.00 ft

Design Moments and Uniform Live Load
Ultimate moment in minor-axis, \( M_{yu} \) = 266.00 kip*ft
Ultimate shear force, \( Q_{xu} \) = 20.00 kips

Section Details
sec.: SEL("AISC/W";NAME; ) = W24X131
depth, \( d \) = TAB("AISC/W";d;NAME=sec.) = 24.50 in
Web th., \( t_w \) = TAB("AISC/W";t\(_w\);NAME=sec.) = 0.60 in
Flange width, \( b_f \) = TAB("AISC/W";b\(_f\);NAME=sec.) = 12.90 in
Flange th., \( t_f \) = TAB("AISC/W";t\(_f\);NAME=sec.) = 0.96 in
Plastic sec. modulus, \( Z_y \) = TAB("AISC/W";Z\(_y\);NAME=sec.) = 81.50 in\(^3\)
Elastic sec. modulus, \( S_y \) = TAB("AISC/W";S\(_y\);NAME=sec.) = 53.00 in\(^3\)
AISC Specification Eqn. (F6-1):
Yielding Moment, \( M_p \) = MIN (\( Z_y \times F_y \times 1/12 \); 1.6/12\( S_y \times F_y \)) = 340 kip*ft
Element Classification
Flanges:
b_f/2t_f, \lambda_f = \text{TAB}(*AISC/W*;b_f/2t_f;NAME=sec.) = 6.7

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

\lambda_{pf} = 0.38*\sqrt{(E/F_y)} = 9.2
\lambda_{rf} = 1.0*\sqrt{(E/F_y)} = 24.1

Fl_Class = IF(\lambda_f <= \lambda_{pf} ; "Compact" ; "Non-Compact") = Compact

Because the beam bent about the minor-axis, the available strength is governed by AISC Specification Sections F6.1 and F6.2. The nominal flexural moment is calculated as follows, satisfying the condition of compression Flange Local Buckling:

M_{n1} = IF(Fl_Class="Compact"; M_p; (M_p-(0.7*F_y*S_y*1/12)*((\lambda_f - \lambda_{pf})/(\lambda_{rf} - \lambda_{pf})))) = 340 kip*ft

Check The Available Flexure Strength

\phi M_n = 0.90*MIN(M_p;M_{n1}) = 306 kip*ft

Safety = IF(\phi M_n >= M_{yu};"Safe","Unsafe") = Safe

Moment ratio = M_{yu}/\phi M_n = 0.87

Check Shear Strength Section G (AISC Spec.)

Calculate A_w. (Multiply by 2 for both shear resisting elements.)

A_w = 2*b_f*t_f = 24.77 in^2

Calculate Cv: (Eqns. G2-2, G2-3, G2-4 and G2-5)

k_v = 1.20

\psi_{f1} = 1.1*\sqrt{(k_v*E/F_y)} = 29.0

\psi_{f2} = 1.37*\sqrt{(k_v*E/F_y)} = 36.1

C_v = IF(\lambda_f <= \psi f_{11};1; IF(\lambda_f > \psi f_{11} AND \lambda_f <= \psi f_{21};\psi f_{11}/\lambda_f;1.51*kv*E/(Fy*(f^2)))) = 1.0

From AISC Specification Section G2.1, the available shear strength is:

\phi_v = 0.90

Nominal shear strength, V_n = 0.6*F_y*A_w*C_v = 743 kips

Design shear, \phi V_n = \phi_v * V_n = 669 kips

Shear_safety = IF(V_n > Q_{yu}; "Safe" ; "Unsafe") = Safe
Design Summary

\[ \Phi M_n = 0.90 \cdot \text{MIN}(M_p; M_{n1}) \]
\[ \text{Safety} = \text{IF}(\Phi M_n \geq M_{yu}; \text{"Safe"}; \text{"Unsafe"}) \]
\[ \text{Moment ratio} = \frac{M_{yu}}{\Phi M_n} \]
\[ \Phi V_n \cdot V_n = \Phi V_n \cdot V_n \]
\[ \text{Shear_safety} = \text{IF}(V_n > Q_{yu}; \text{"Safe"}; \text{"Unsafe"}) \]

\[ \Phi M_n = 306 \text{ kip*ft} \]
\[ \text{Safety} = \text{Safe} \]
\[ \text{Moment ratio} = 0.87 \]
\[ \Phi V_n \cdot V_n = 669 \text{ kips} \]
\[ \text{Shear_safety} = \text{Safe} \]
Design of W-Shapes Subjected to Tension Force and Bending Moments

Materials

Grade: SEL("Material/ASTM"; NAME; ) = A992

$F_y$ = TAB("Material/ASTM"; $F_y$; NAME=Grade) = 50 ksi

$E$ = 29000 ksi

Beam Length

Unsupported length, $L_b$ = 30.00 ft

$kL_{in}$ = 14.00 ft

$kL_{out}$ = 30.00 ft

($kL_{in}$ and $kL_{out}$ are the strong and weak/torsional unbraced lengths, respectively)

From Table 3-1 (AISC), $C_{b1}$ = 1.14

Given Straining Actions

Dead Load:

$T_D$ = 29.0 kips

$M_{xD}$ = 32.0 kip*ft

$M_{yD}$ = 11.3 kip*ft

Live Load:

$T_L$ = 87.0 kips

$M_{xL}$ = 96.0 kip*ft

$M_{yL}$ = 33.8 kip*ft

Ultimate Tension Force and Bending Moments

$T_u$ = $1.2 \cdot T_D + 1.6 \cdot T_L$ = 174.0 kips

$M_{ux}$ = $1.2 \cdot M_{xD} + 1.6 \cdot M_{xL}$ = 192.0 kip*ft

$M_{uy}$ = $1.2 \cdot M_{yD} + 1.6 \cdot M_{yL}$ = 67.6 kip*ft
Chapter 3: Steel Design

W-Shape Subjected to Tension Force and Bending Moments

Section Details

- **sec.**: SEL("AISC/W";NAME; ) = W14X82
- **depth, d**: TAB("AISC/W";d;NAME=sec.) = 14.30 in
- **Web th., t_w**: TAB("AISC/W";t_w;NAME=sec.) = 0.51 in
- **Flange width, b_f**: TAB("AISC/W";b_f;NAME=sec.) = 10.10 in
- **Flange th., t_f**: TAB("AISC/W";t_f;NAME=sec.) = 0.85 in
- **Gross Area, A**: TAB("AISC/W";A;NAME=sec.) = 24.00 in²
- **I_x**: TAB("AISC/W";I_x;NAME=sec.) = 881.00 in⁴
- **I_y**: TAB("AISC/W";I_y;NAME=sec.) = 148.00 in⁴

 *(I_x and I_y are the moment of inertia about x-and y-axes, respectively)*

- **Plastic sec. modulus, Z_x**: TAB("AISC/W";Z_x;NAME=sec.) = 139.00 in³
- **Elastic sec. modulus, S_x**: TAB("AISC/W";S_x;NAME=sec.) = 123.00 in³
- **Plastic sec. modulus, Z_y**: TAB("AISC/W";Z_y;NAME=sec.) = 44.80 in³
- **Elastic sec. modulus, S_y**: TAB("AISC/W";S_y;NAME=sec.) = 29.30 in³
- **r_x**: TAB("AISC/W";r_x;NAME=sec.) = 6.05 in
- **r_y**: TAB("AISC/W";r_y;NAME=sec.) = 2.48 in

 *(r_x and r_y are the radius of gyration about x- and y-axis, respectively)*

- **Torsional constant, J**: TAB("AISC/W";J;NAME=sec.) = 5.07 in⁴
- **r_ts**: TAB("AISC/W";r_ts;NAME=sec.) = 2.85 in
- **h_o**: TAB("AISC/W";h_o;NAME=sec.) = 13.40 in

 *(r_ts is the Effective radius of gyration for the L.T.B. and h_o is distance between C.L. of flanges)*

AISC Specification Eqn. (F6-1), the yielding moment in minor axis (M_y):

\[ M_y = \min (Z_y F_y * 1/12; 1.6/12 * S_y F_y) = 187 \text{ kip*ft} \]

**Slenderness Check (According to section E2)**

For members designed on the basis of compression, the slenderness ratio KL/r should not exceed 300.

\[ \lambda_x = \frac{kL_in}{r_x} = 27.8 \]
\[ \lambda_y = \frac{kL_out}{r_y} = 145.2 \]

Then, the governed slenderness (\( \lambda_{\text{max}} \)):

\[ \lambda_{\text{max}} = \max (\lambda_x; \lambda_y) = 145.2 \]

*Slenderness_check = IF(\( \lambda_{\text{max}} \leq 300; \text{"Safe"}; \text{"Unsafe"} \)) = Safe*
Nominal Tensile Strength

From AISC Specification Section D2(a), the nominal tensile strength due to tensile yielding on the gross section is:

\[ T_n = F_y \cdot A = 1200.0 \text{ kips} \]

Nominal Flexural Strength about x-x Axis

Yielding: from AISC specification section F2.1, the nominal flexural strength due to yielding (plastic moment) is:

\[ M_{px} = Z_x \cdot F_y \cdot 1/12 = 579 \text{ kip*ft} \]

Lateral torsional buckling (LTB): the limiting lengths \( L_p \) and \( L_r \) are determined according to the AISC spec. eqns. F2-5 and F2-6, as follows:

\[ L_p = 1.76 \cdot r_y \cdot \sqrt{(E/F_y)/12} = 8.76 \text{ ft} \]
\[ L_{r1} = \sqrt{\frac{J \cdot 1.0}{S_x \cdot h_o}} = 0.06 \]
\[ L_{r2} = \sqrt{1 + \left(6.76 \cdot \frac{0.7 \cdot F_y \cdot S_x \cdot h_o}{E \cdot J \cdot 1.0}\right)^2} = 1.42 \]
\[ L_r = 1.95/12 \cdot r_{ts} \cdot \sqrt{E \cdot 0.7 \cdot F_y} - L_{r1} \cdot L_{r2} = 32.69 \text{ ft} \]

Case = IF(L_b > L_r; "ELTB"; IF(L_b < L_p; "No LTB"; "InLTB")) = InLTB

("ELTB" refers to elastic lateral torsional buckling and "InLTB" refers to inelastic lateral torsional buckling).

The lateral torsional buckling modification factor, \( C_b \):

\[ T_{ey} = \frac{2}{\pi} \cdot \frac{E \cdot I_y}{(L_b \cdot 12)^2} = 326.9 \text{ kips} \]
\[ C_b = C_{b1} \cdot \sqrt{1 + \frac{T_u}{T_{ey}}} = 1.41 \]

According to the AISC Spec. Eqn. F2-2:

\[ M_{1a} = M_{px} \cdot 0.7 \cdot 1/12 \cdot F_y \cdot S_x = 220 \text{ kip*ft} \]
\[ M_{1} = \text{MIN}(M_{px}; C_b \cdot (M_{px} - M_{1a} \cdot (L_b - L_p)/(L_r - L_p))) = 541 \text{ kip*ft} \]
Chapter 3: Steel Design
W-Shape Subjected to Tension Force and Bending Moments

According to the AISC Spec. Eqn. F2-4:

\[ F_{cr} = \frac{C_b \cdot \pi \cdot E}{\left( \frac{L_b + 0.01 \cdot 12}{r_{ts}} \right)^2} = 25.28 \text{ ksi} \]

\[ F_{cr,mod} = \sqrt{1 + \frac{0.078 \cdot J \cdot 1.0 \cdot \left( \frac{L_b \cdot 12}{r_{ts}} \right)^2}{S_x \cdot h_o}} = 2.20 \text{ ksi} \]

According to the AISC Spec. Eqns. F2-3:

\[ M_2 = \text{MIN}(M_{px} \cdot F_{cr} \cdot S_x / 12 \cdot F_{cr,mod}) = 570 \text{ kip*ft} \]

According to the AISC Spec. Eqn. F2-2:

\[ M_{nx2} = \text{IF (Case="No LTB";M_{px};IF(Case="InLTB";M_1;M_2))} = 541 \text{ kip*ft} \]

Element Classification

(1) Web:

\[ h/tw, \lambda_w = \text{TAB("AISC/W";h/t_w;NAME=sec.)} = 22.40 \]

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

Web Class = IF(\( \lambda_w \leq 3.76 \sqrt{E/F_y} \);"Compact";"Non-Compact") = Compact

(2) Comp. flange:

\[ b/2t_f, \lambda_f = \text{TAB("AISC/W";b/2t_f;NAME=sec.)} = 5.92 \]

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

\[ \lambda_{pf} = 0.38 \sqrt{E/F_y} = 9 \]

\[ \lambda_{rf} = 1.00 \sqrt{E/F_y} = 24 \]

Fl_Class = IF(\( \lambda_f \leq \lambda_{pf} \);"Compact";IF(\( \lambda_f > \lambda_{rf} \);"Slender";"Non-Compact")) = Compact

The available strength provided by AISC Specification Sections F3.1, F3.2, F6.1 and F6.2, the nominal flexural moments in strong/weak axes are calculated as follows, satisfying the condition of compression Flange Local Buckling:

\[ M_{nx1a} = M_{px} \cdot 0.7 \cdot F_y \cdot S_x \cdot 1/12 = 220 \text{ kip*ft} \]

\[ M_{nx1} = \text{IF(Fl_Class="Compact"; M_{px}; M_{px} \cdot M_{nx1a}(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}))} = 579 \text{ kip*ft} \]

\[ M_{ny1a} = M_{py} \cdot 0.7 \cdot F_y \cdot S_y \cdot 1/12 = 102 \text{ kip*ft} \]

\[ M_{ny1} = \text{IF(Fl_Class="Compact"; M_{py}; M_{py} \cdot M_{ny1a}(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}))} = 187 \text{ kip*ft} \]
Chapter 3: Steel Design

W-Shape Subjected to Tension Force and Bending Moments

Design Flexure Moment in Major/Minor Axes

\[
\begin{align*}
M_{nx} &= \text{MIN}(M_{px}; M_{nx1}; M_{nx2}) = 541 \text{ kip*ft} \\
M_{ny} &= \text{MIN}(M_{py}; M_{ny1}) = 187 \text{ kip*ft}
\end{align*}
\]

Calculate The Available Flexural and Axial Strengths

\[
\begin{align*}
\Phi_b &= 0.90 \\
\Phi_t &= 0.90 \\
T_c &= \Phi_t \cdot T_n = 1080 \text{ kips} \\
M_{cx} &= \Phi_b \cdot M_{nx} = 487 \text{ kip*ft} \\
M_{cy} &= \Phi_b \cdot M_{ny} = 168 \text{ kip*ft}
\end{align*}
\]

Interaction of Tension and Flexure

Check limit for AISC Specification Equation H1-1a.

\[
\begin{align*}
\text{Tension\_ratio, } t &= \frac{T_u}{T_c} = 0.16 \\
\text{Moment\_ratio, } m &= \frac{M_{ux} + M_{uy}}{M_{cx} + M_{cy}} = 0.80 \\
\text{Safety\_ratio, } r &= \text{IF}(t \geq 0.2; t+8/9*m; t/2+m) = 0.88 \\
\text{Safety} &= \text{IF}(r \leq 1; \text{"Safe"};\text{"Unsafe"}) = \text{Safe}
\end{align*}
\]

Design Summary

\[
\begin{align*}
T_u &= 1.2*T_D + 1.6*T_L = 174.0 \text{ kips} \\
M_{ux} &= 1.2*M_{x_D} + 1.6*M_{x_L} = 192.0 \text{ kip*ft} \\
M_{uy} &= 1.2*M_{y_D} + 1.6*M_{y_L} = 67.6 \text{ kip*ft} \\
T_c &= \Phi_t \cdot T_n = 1080 \text{ kips} \\
M_{cx} &= \Phi_b \cdot M_{nx} = 487 \text{ kip*ft} \\
M_{cy} &= \Phi_b \cdot M_{ny} = 168 \text{ kip*ft} \\
\text{Slenderness\_check} &= \text{IF}(\lambda_{\text{max}} \leq 300; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe} \\
\text{Safety\_ratio, } r &= \text{IF}(t \geq 0.2; t+8/9*m; t/2+m) = 0.88 \\
\text{Safety} &= \text{IF}(r \leq 1; \text{"Safe"};\text{"Unsafe"}) = \text{Safe}
\end{align*}
\]
Design of W-Shapes Subjected to Axial Compression

**Materials**

Grade: SEL("Material/ASTM": NAME; ) = A992

\[ F_y = \text{TAB("Material/ASTM":F_y;NAME=Grade)} = 50.00 \]

\[ E = 29000 \text{ ksi} \]

**Buckling Lengths**

\[ kL_{in} = 30.00 \text{ ft} \]

\[ kL_{out} = 15.00 \text{ ft} \]

\((kL_{in} \text{ and } kL_{out} \text{ are unbraced lengths for the strong- and weak-axes})\)

**Axial Loads**

Dead load, \( P_D = 140 \text{ kips} \)

Live load, \( P_L = 420 \text{ kips} \)

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

\[ \text{Ultimate load, } P_u = 1.2P_D + 1.6P_L = 840 \text{ kips} \]

**Section Details**

sec.: SEL("AISC/W":NAME; ) = W14X90

\[ \text{depth, } d = \text{TAB("AISC/W":d;NAME=sec.) = 14.00 \text{ in}} \]

\[ \text{Web th., } t_w = \text{TAB("AISC/W":t_w;NAME=sec.) = 0.44 \text{ in}} \]

\[ \text{Flange width, } b_f = \text{TAB("AISC/W":b_f;NAME=sec.) = 14.50 \text{ in}} \]

\[ \text{Flange th., } t_f = \text{TAB("AISC/W":t_f;NAME=sec.) = 0.71 \text{ in}} \]

\[ \text{Area, } A = \text{TAB("AISC/W":A;NAME=sec.) = 26.50 \text{ in}^2} \]

\[ r_x = \text{TAB("AISC/W":r_x;NAME=sec.) = 6.14 \text{ in}} \]

\[ r_y = \text{TAB("AISC/W":r_y;NAME=sec.) = 3.70 \text{ in}} \]

\( (r_x \text{ and } r_y \text{ are the radius of gyration about } x- \text{ and } y-\text{axes, respectively}) \)
Element Classification (According to Table B4-1)

(1) Web:
\[ h/t_w, \lambda_w = \frac{h}{t_w} \]  
\[ \lambda_w = \text{TAB(“AISC/W”;h/t_w;NAME=sec.)} = 25.90 \]  
According to AISC Specification Table B4.1 Case 10, the limiting width-to-thickness ratio for non-compact web is:
\[ \text{Web}_\text{Class} = \begin{cases} \text{Non-Compact} & \text{if } \lambda_w \leq 1.49 \times \sqrt{(E/F_y)}; \\ \text{Slender} & \text{otherwise} \end{cases} \]

(2) Flanges:
\[ b_f/2t_f, \lambda_f = \frac{b_f}{2t_f} \]  
\[ \lambda_f = \text{TAB(“AISC/W”;b_f/2t_f;NAME=sec.)} = 10.20 \]  
According to AISC Specification Table B4.1 Case 4, the limiting width-to-thickness ratio for non-compact flange is:
\[ k_c = \min(\max(4/\sqrt{\lambda_f}; 0.35); 0.76) = 0.76 \]  
\[ \lambda_{rf} = 0.64 \times \sqrt{(k_c \times E/F_y)} = 13 \]  
\[ \text{Fl}_\text{Class} = \begin{cases} \text{Non-Compact} & \text{if } \lambda_f \leq \lambda_{rf}; \\ \text{Slender} & \text{otherwise} \end{cases} \]

Slenderness Check (According to Section E2)

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.
\[ \lambda_x = \frac{k L_{in}}{r_x} \times 12 = 58.6 \]  
\[ \lambda_y = \frac{k L_{out}}{r_y} \times 12 = 48.6 \]  
Then, the governed slenderness (\( \lambda_{\text{max}} \)):
\[ \lambda_{\text{max}} = \max(\lambda_x; \lambda_y) = 58.6 \]

Critical Stresses

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:
- Calculate the elastic critical buckling stress, \( F_e \):
\[ F_e = \frac{\pi^2 \times E}{\lambda_{\text{max}}^2} = 83.3 \text{ ksi} \]
- Calculate the flexural buckling stress, \( F_{cr} \) (Eqns. E3-2 and E3-3):
\[ \lambda_1 = 4.71 \times \sqrt{(E/F_y)} = 113 \]  
\[ F_{cr} = \begin{cases} \text{IF}(\lambda_{\text{max}} \leq \lambda_1; 0.658 \times (F_y/F_e) \times F_y \times 0.877 \times F_e) & \text{if } \lambda_{\text{max}} \leq \lambda_1; \\ 0 & \text{otherwise} \end{cases} \]  
\[ F_{cr} = 38.9 \text{ ksi} \]
Nominal Compressive Strength (Eqn. E3-1)

\[ P_n = F_{cr} * A = 1031 \text{ kips} \]

\[ \phi_v = 0.90 \]

\[ \phi_v P_n = \phi_v * P_n = 928 \text{ kips} \]

Compressive stress safety \( S_s \):

\[ S_s = \text{IF}(\phi_v * P_n > P_u; "Safe"; "Unsafe") = \text{Safe} \]

Stress ratio:

\[ P_u / (\phi_v * P_n) = 0.91 \]

Design Summary

Ultimate load, \( P_u \):

\[ 1.2*P_D + 1.6*P_L = 840 \text{ kips} \]

Design load, \( \phi_v P_n \):

\[ \phi_v * P_n = 928 \text{ kips} \]

Stress ratio:

\[ P_u / (\phi_v * P_n) = 0.91 \]

\[ S_s = \text{IF}(\phi_v * P_n > P_u; "Safe"; "Unsafe") = \text{Safe} \]
Design of WT-Shapes Subjected to Compression Axial Force

Materials

Grade: SEL("Material/ASTM";NAME; ) = A992

\( F_y = \) TAB("Material/ASTM";\( F_y \);NAME=Grade) = 50 ksi

\( E = \) 29000 ksi

\( G = \) 11200 ksi

Buckling Lengths

\( k_{L_{in}} = \) 20.00 ft

\( k_{L_{out}} = \) 20.00 ft

\( k_z L = \) 20.00 ft

\( (k_{L_{in}} \text{ and } k_{L_{out}} \text{ are unbraced lengths for the strong- and weak- axes, respectively; } k_z L \text{ is the torsional unbraced length}) \)

Axial Loads

Axial dead load, \( P_D = \) 6 kips

Axial Live load, \( P_L = \) 18 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, \( P_u = 1.2*P_D + 1.6*P_L \) = 36.0 kips

Section Details

sec.: SEL("AISC/WT"; NAME; ) = WT7X15

Depth, \( d = \) TAB("AISC/WT";\( d \);NAME=sec.) = 6.92 in

Stem thickness, \( t_w = \) TAB("AISC/WT";\( t_w \);NAME=sec.) = 0.270 in

Flange width, \( b_f = \) TAB("AISC/WT";\( b_f \);NAME=sec.) = 6.73 in

Flange thickness, \( t_f = \) TAB("AISC/WT";\( t_f \);NAME=sec.) = 0.385 in

Gross area, \( A = \) TAB("AISC/WT";\( A \);NAME=sec.) = 4.4 in\(^2\)
Chapter 3: Steel Design

WT-Shapes in Axial Compression

$I_x =$ \text{TAB("AISC/WT";I_x;NAME=sec.)} = 19 \text{ in}^4

$I_y =$ \text{TAB("AISC/WT";I_y;NAME=sec.)} = 10 \text{ in}^4

($I_x$ and $I_y$ are the moment of inertia about x- and y-axes, respectively)

$r_x =$ \text{TAB("AISC/WT";r_x;NAME=sec.)} = 2.07 \text{ in}

$r_y =$ \text{TAB("AISC/WT";r_y;NAME=sec.)} = 1.49 \text{ in}

($r_x$ and $r_y$ are the radius of gyration about x- and y-axis, respectively)

$Torsion\ constant, J =$ \text{TAB("AISC/WT";J;NAME=sec.)} = 0.19 \text{ in}^4

$Q_s =$ \text{TAB("AISC/WT";Q_s;NAME=sec.)} = 0.61

$y =$ \text{TAB("AISC/WT";y;NAME=sec.)} = 1.58 \text{ in}

($Q_s$ is a reduction factor for unstiffned elements and $y$ is the distance to the N.A.)

Element Classification

(1) Flanges:

$\frac{b}{2t_f}, \lambda_f =$ $\frac{b}{2t_f} = 8.74$

Determine the flange limiting slenderness ratio, $\lambda_f$, from AISC Specification Table B4.1a case 2:

$\lambda_f =$ $0.56 \sqrt{\frac{E}{F_y}} = 13.5$

Fl_Class$ = \text{IF}(\lambda_f \leq \lambda_{rf};"Non-Compact";"Slender") = \text{Non-Compact}$

(2) Web:

$\frac{d}{t_w}, \lambda_w =$ $d/t_w = 25.6$

Determine the slender web limit from AISC Specification Table B4.1a case 4:

$\lambda_{rw}$ = $0.75 \sqrt{\frac{E}{F_y}} = 18.06$

Web_Class$ = \text{IF}(\lambda_w \leq \lambda_{rw};"Non-compact"; "Slender") = \text{Slender}$

$Q =$ $Q_s = 0.610$

Slenderness check:

For members designed on the basis of compression, the slenderness ratio $KL/r$ preferably should not exceed 200.

$\lambda_x =$ $\frac{KL_{in} * 12}{r_x} = 115.9$

$\lambda_y =$ $\frac{KL_{out} * 12}{r_y} = 161.1$

$\lambda_{max} =$ $\text{MAX}(\lambda_x; \lambda_y) = 161.1$
X-X Axis Critical Elastic Flexural Buckling Stress:

\[ F_{ex} = \frac{2 \pi \cdot E}{\lambda_x} \]

= 21.3 ksi

Critical Elastic Torsional and Flexural-Torsional Buckling Stress:

\[ F_{ey} = \frac{2 \pi \cdot E}{\lambda_y} \]

= 11.0 ksi

**Torsional Parameters**

The shear center for a T-shaped section is located on the axis of symmetry at the mid-depth of the flange.

\[ x_0 = 0.0 \text{ in} \]

\[ y_0 = y - \frac{t_f}{2} = 1.39 \text{ in} \]

According to the AISC Specification Eqn. E4-11:

\[ r_o = \sqrt{(x_0^2 + y_0^2) \cdot \frac{l_x + l_y}{A}} \]

= 2.92 in

According to the AISC Specification Eqn. E4-10:

\[ H = 1 - \frac{x_0^2 + y_0^2}{r_o^2} \]

= 0.77 in

According to the AISC Specification Eqn. E4-9:

\[ F_{ez} = \left( \frac{2 \pi \cdot E \cdot C_w}{(k_z \cdot L)^2} + GJ \right) \cdot \frac{1}{A \cdot r_o^2} \]

Omit term with \( C_w \) per User Note at end of AISC Specification Section E4.

\[ F_{ez} = \frac{G \cdot J}{A \cdot r_o^2} \]

= 56.72 ksi

According to the AISC Specification Eqn. E4-5:

\[ F_{e2} = \frac{F_{ey} + F_{ez}}{2 \cdot H} \cdot \left( 1 - \sqrt{1 - \frac{4 \cdot F_{ey} \cdot F_{ez} \cdot H}{(F_{ey} + F_{ez})^2}} \right) \]

= 10.5 ksi

**Governed Critical Elastic Buckling Stress**

\[ F_e = \text{MIN} \left( F_{ex}, F_{ey}, F_{e2} \right) \]

= 10.5 ksi
Buckling Stress for The Section

Determine whether AISC Specification Equation E7-2 or E7-3 applies.

\[ F_{cr} = 0.44*Q*F_y \]

\[ F_{cr} = \text{IF}(F_e \geq F_{cr}; Q*0.658*(F_y/F_e)^{0.658}*F_y;0.877*F_e) \]

\[ = 13.4 \text{ ksi} \]

Nominal Compressive Strength

\[ P_n = F_{cr}*A \]

\[ = 40.5 \text{ kips} \]

\[ \Phi_v = 0.90 \]

\[ \Phi_{v}P_n = \Phi_v*P_n \]

\[ = 36.5 \text{ kips} \]

Compressive stress safety (S_s):

\[ S_s = \text{IF}(\Phi_vP_n>P_u;"Safe";"Unsafe") \]

\[ = \text{Safe} \]

Stress ratio:

\[ \frac{P_u}{\Phi_v*P_n} \]

\[ = 0.99 \]

Design Summary

Ultimate load, \( P_u = 1.2*P_D+1.6*P_L \)

\[ = 36.0 \text{ kips} \]

Design load, \( \Phi_{v}P_n = \Phi_v*P_n \)

\[ = 36.5 \text{ kips} \]

Stress ratio:

\[ \frac{P_u}{\Phi_v*P_n} \]

\[ = 0.99 \]

\[ S_s = \text{IF}(\Phi_v*P_n>P_u;"Safe";"Unsafe") \]

\[ = \text{Safe} \]
Design of Built-Up W-Shapes with Slender Elements Subjected to Compression Axial Force

Materials

<table>
<thead>
<tr>
<th>Grade</th>
<th>SEL(&quot;Material/ASTM&quot;;NAME; )</th>
<th>= A992</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_y$</td>
<td>TAB(&quot;Material/ASTM&quot;;F;y;NAME=Grade)</td>
<td>= 50 ksi</td>
</tr>
<tr>
<td>$E$</td>
<td>29000 ksi</td>
<td></td>
</tr>
<tr>
<td>$G$</td>
<td>11200 ksi</td>
<td></td>
</tr>
</tbody>
</table>

Buckling Lengths

- $k_{L_{in}}$ = 30.00 ft
- $k_{L_{out}}$ = 15.00 ft
- $k_{zL}$ = 15.00 ft

($k_{L_{in}}$ and $k_{L_{out}}$ are unbraced lengths for the strong- and weak- axes, respectively; $k_{zL}$ is the torsional unbraced length)

Axial Loads

- Dead load, $P_D$ = 140 kips
- Live load, $P_L$ = 200 kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

$$P_u = 1.2\times P_D + 1.6\times P_L = 488 \text{ kips}$$

Section Details

- Web height, $h$ = 15.0 in
- Web th., $t_w$ = 0.25 in
- Flange width, $b_f$ = 8.00 in
- Flange th., $t_f$ = 1.00 in
Built-Up Section Properties (ignoring fillet welds):

Area, $A = h^*t_w + 2*b^*_f*t_f = 19.75 \text{ in}^2$

$I_x = 2*(b^*_f*t_f)^*(t_f/2+h/2)^2 + \frac{t_w^*h^3}{12} + \frac{b^*_f*(t_f^3)}{2} * \frac{2}{12} = 1096 \text{ in}^4$

$I_y = \frac{2*b^*_f^3}{12} * t_f^3 + \frac{h^*t_w^3}{12} = 85.35 \text{ in}^4$

$r_x = \sqrt{\frac{I_x}{A}} = 7.45 \text{ in}$

$r_y = \sqrt{\frac{I_y}{A}} = 2.08 \text{ in}$

Slenderness Check

For members designed on the basis of compression, the slenderness ratio $KL/r$ preferably should not exceed 200.

$\lambda_x = \frac{kL_{in}}{r_x} * 12 = 48.3$

$\lambda_y = \frac{kL_{out}}{r_y} * 12 = 86.5$

Then, the governed slenderness ($\lambda_{max}$):

$\lambda_{max} = \text{MAX}(\lambda_x; \lambda_y) = 86.5$

Elastic Flexural Buckling Stress

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:

$F_{e1} = \frac{\pi^2 * E}{2 * \lambda_{max}^2} = 38.3 \text{ ksi}$

Elastic Critical Torsional Buckling Stress

From the User Note in AISC Specification Section E4,

$h_o = h + t_f = 16 \text{ in}$

$C_w = \frac{l_y^*h_o^2}{4} = 5462 \text{ in}^6$

From AISC Design Guide 9, Equation 3.4,

$J = \frac{2*b^*_f^3}{3} * t_f^3 + \frac{h^*t_w^3}{3} = 5.41 \text{ in}^4$
According to AISC Specification (Eqn. E4-4),

\[
F_{e2} = \frac{\pi^2 E C_w}{(kzL \times 12)^2} \left( 1 + G J \right) x^2 \left( 1 + I_y \right) = 92.1 \text{ ksi}
\]

Elastic Governor Stress

\[
F_e = \min(F_{e1}, F_{e2}) = 38.3 \text{ ksi}
\]

Element Classification

1. Flanges: Check for slender flanges using AISC Specification Table B4.1a, then determine Qs, the unstiffened element (flange) reduction factor using AISC Specification Section E7.1. Calculate \( k_c \) using AISC Specification Table B4.1b note [a].

\[
k_c = \min(\max(4/\sqrt{h/t_w}; 0.35) ; 0.76) = 0.52
\]

\[
b_f/2t_f, \lambda_f = b_f/(2t_f) = 4.00
\]

Determine the flange limiting slenderness ratio, \( \lambda_{rf} \), from AISC Specification Table B4.1a case 2

\[
\lambda_{rf} = 0.64 \times \sqrt{k_c E/F_y} = 11.11
\]

Fl_class = IF(\( \lambda_f \leq \lambda_{rf} \), "Non-Compact"; "Slender") = Non-Compact

Calculate Qs, according to the AISC Specification Eqns. E7-4, E7-5 and E7-6

\[
\lambda_{rf1} = 1.17 \times \sqrt{k_c E/F_y} = 20.32
\]

\[
Q_{s1} = 0.9 \times k_c / (F_y \times \lambda_f^2) = 16.97
\]

\[
Q_{s2} = 1.415 - 0.65 \times \lambda_f \times \sqrt{F_y/(E k_c)} = 1.27
\]

\[
Q_s = \text{IF}(\lambda_f < \lambda_{rf1}, 1; \text{IF}(\lambda_f > \lambda_{rf1}, Q_{s1}, Q_{s2}))) = 1.00
\]

Web: Check for a slender web, then determine Qa, the stiffened element (web) reduction factor using AISC Specification, Section E7.2.

\[
\frac{h}{t_w}, \lambda_w = h/t_w = 60.00
\]

Determine the slender web limit from AISC Specification Table B4.1a case 5

\[
\lambda_{rw} = 1.49 \times \sqrt{E/F_y} = 35.88
\]

Web_class = IF(\( \lambda_w \leq \lambda_{rw} \), "Non-compact"; "Slender") = Slender

\[
Q_a = \frac{A_e}{A_g}
\]

where \( A_e \) is the effective area based on the reduced effective width (\( b_e \)). For AISC Specification Equation E7-17, take \( f \) as \( F_{cr} \) with \( F_{cr} \) calculated based on \( Q = 1.0 \). Select between AISC Specification Equations E7-2 and E7-3 based on \( KL/r_y \).

\[
\lambda_1 = 4.71 \times \sqrt{\frac{E}{F_y}} = 113
\]
Calculate the flexural buckling stress, \( F_{cr} \):

\[
F_{cr1} = \text{IF}(\lambda_{\text{max}} \leq \lambda_1; 0.658 \left( \frac{F_y}{F_e} \right) * F_y : 0.877 * F_e) = 29.0 \text{ ksi}
\]

\[
b_e = \text{MIN}(1.92 \cdot t_w \cdot \sqrt{\frac{E}{F_{cr1}}} \cdot (1 - 0.34 \cdot \frac{\lambda_w}{\sqrt{F_{cr1}}}); h) = 12.5 \text{ in}
\]

(Note that \( b_e \) should be less than the web height)

\[
A_e = b_e \cdot t_w + 2 \cdot b_f \cdot t_f = 19.1 \text{ in}^2
\]

\[
Q_a = \frac{A_e}{A} = 0.967
\]

\[
Q = Q_s \cdot Q_a = 0.967
\]

**Flexural Buckling Strength**

Determine another time whether AISC Specification Equation E7-2 or E7-3 applies.

\[
\lambda_2 = 4.71 \cdot \sqrt{\frac{E}{Q \cdot F_y}} = 115
\]

\[
F_{cr2} = \text{IF}(\lambda_{\text{max}} \leq \lambda_1; Q \cdot 0.658 \left( \frac{F_y}{Q \cdot F_e} \right) * F_y : 0.877 * F_e) = 28.5 \text{ ksi}
\]

**Nominal Compressive Strength**

\[
P_n = F_{cr2} \cdot A = 563 \text{ kips}
\]

\[
\phi_v = 0.90
\]

\[
\phi_v \cdot P_n = \phi_v \cdot P_n = 507 \text{ kips}
\]

Compressive stress safety (\( S_s \)):

\[
S_s = \text{IF}(\phi_v \cdot P_n > P_u;"Safe","Unsafe") = \text{Safe}
\]

\[
\text{Stress\_ratio} = \frac{P_u}{\phi_v \cdot P_n} = 0.96
\]

**Design Summary**

Ultimate load, \( P_u \) = \( 1.2 \cdot P_D + 1.6 \cdot P_L \) = 488 kips

\[
\phi_v \cdot P_n = \phi_v \cdot P_n = 507 \text{ kips}
\]

\[
\text{Stress\_ratio} = \frac{P_u}{\phi_v \cdot P_n} = 0.96
\]

\[
S_s = \text{IF}(\phi_v \cdot P_n > P_u;"Safe","Unsafe") = \text{Safe}
\]
Design of W-Shapes Subjected to Compression Force and Bending Moment

Materials
Grade: SEL("Material/ASTM"; NAME; ) = A992
Fy = TAB("Material/ASTM";F_y;NAME=Grade) = 50 ksi
E = 29000 ksi

Beam Length and Cb
Unsupported length, L_b = 14.00 ft
k_L_in = 14.00 ft
k_L_out = 14.00 ft
(k_L_in and k_L_out are strong- and weak/torsional- axes unbraced length, respectively)
From Table 3-1 (AISC ), C_b = 1.00

Ultimate Compression Force and Bending Moments
(obtained from a second-order analysis that includes P- δ effects)
P_u = 200.0 kips
M_u_x = 50.0 kip*ft
M_u_y = 50.0 kip*ft
Section Details

sec.: SEL("AISC/W";NAME; ) = W14X68
depth, d = TAB("AISC/W";d;NAME=sec.) = 14.00 in
Web th., t_w = TAB("AISC/W";t_w;NAME=sec.) = 0.41 in
Flange width, b_f = TAB("AISC/W";b_f;NAME=sec.) = 10.00 in
Flange th., t_f = TAB("AISC/W";t_f;NAME=sec.) = 0.72 in
Gross Area, A = TAB("AISC/W";A;NAME=sec.) = 20.00 in^2
I_x = TAB("AISC/W";I_x;NAME=sec.) = 722.00 in^4
I_y = TAB("AISC/W";I_y;NAME=sec.) = 121.00 in^4
(I_x and I_y are the moment of inertia about x-and y-axes, respectively)

Plastic sec. modulus, Z_x = TAB("AISC/W";Z_x;NAME=sec.) = 115.00 in^3
Elastic sec. modulus, S_x = TAB("AISC/W";S_x;NAME=sec.) = 103.00 in^3
Plastic sec. modulus, Z_y = TAB("AISC/W";Z_y;NAME=sec.) = 36.90 in^3
Elastic sec. modulus, S_y = TAB("AISC/W";S_y;NAME=sec.) = 24.20 in^3
Radius of gyration about x-axis, r_x = TAB("AISC/W";r_x;NAME=sec.) = 6.01 in
Radius of gyration about y-axis, r_y = TAB("AISC/W";r_y;NAME=sec.) = 2.46 in
Torsional constant, J = TAB("AISC/W";J;NAME=sec.) = 3.01 in^4
r_t = TAB("AISC/W";r_t;NAME=sec.) = 2.80 in
h_o = TAB("AISC/W";h_o;NAME=sec.) = 13.30 in
(r_t is the Effective radius of gyration for the L.T.B. and h_o is distance between C.L. of flanges)

AISC Specification Eqn. (F2-1):
Yielding Moment in major axis, M_{px} = \frac{Z_x F_y}{12} = 479 kip*ft

AISC Specification Eqn. (F6-1):
Yielding Moment in minor axis, M_{py} = min \left( \frac{Z_y F_y}{12}; \frac{1.6}{12} S_y F_y \right) = 154 kip*ft

Element Classification

(1) Web:
h/t_w, \lambda_w = TAB("AISC/W";h/t_w;NAME=sec.) = 27.50
According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:
Web_Class = IF(\lambda_w \leq 3.76 \sqrt{E/F_y}; "Compact"; "Non-Compact") = Compact

(2) Comp. flange:
b_f/2t_f, \lambda_f = TAB("AISC/W";b_f/2t_f;NAME=sec.) = 6.97
According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

\[ \lambda_{pf} = 0.38 \sqrt{\frac{E}{F_y}} = 9 \]

\[ \lambda_{rf} = 1.00 \sqrt{\frac{E}{F_y}} = 24 \]

\[ \text{Fl Class} = \text{IF}(\lambda_f \leq \lambda_{pf}, "Compact"; \text{IF}(\lambda_f > \lambda_{rf}, "Slender"; "Non-Compact")) = \text{Compact} \]

The available strength provided by AISC Specification Sections F3.1, F3.2, F6.1 and F6.2, the nominal flexural moments in strong/weak axes are calculated as follows, satisfying the condition of compression Flange Local Buckling:

\[ M_{px1} = M_{px} - 0.7 F_y S_x \times \frac{1}{12} = 179 \text{ kip*ft} \]

\[ M_{nx1} = \text{IF(Fl Class="Compact"; } M_{px} - (M_{px} - M_{px1}) \times \left( \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \} = 479 \text{ kip*ft} \]

\[ M_{py1} = M_{py} - 0.7 F_y S_y \times \frac{1}{12} = 83 \text{ kip*ft} \]

\[ M_{ny1} = \text{IF(Fl Class="Compact"; } M_{py} - (M_{py} - M_{py1}) \times \left( \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \} = 154 \text{ kip*ft} \]

**Slenderness Check: (According to Section E2)**

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

\[ \lambda_x = \frac{kL_{in}}{r_x} \times 12 = 28.0 \]

\[ \lambda_y = \frac{kL_{out}}{r_y} \times 12 = 68.3 \]

Then, the governed slenderness (\( \lambda_{max} \)):

\[ \lambda_{max} = \text{MAX}(\lambda_x, \lambda_y) = 68.3 \]

**Critical Stresses**

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:

- Calculate the elastic critical buckling stress, \( F_e \):

\[ F_e = \frac{\pi^2 E}{2 \lambda_{max}^2} = 61.4 \text{ ksi} \]
- Calculate the flexural buckling stress, $F_{cr}$ (Eqns. E3-2 and E3-3):

$$\lambda_1 = \frac{4.71 \sqrt{E}}{F_y} = 113$$

$$F_{cr} = \begin{cases} IF(\lambda_{max} \leq \lambda_1; 0.658 \left( \frac{F_y}{F_e} \right)^*F_y, 0.877*F_e) & = 35.6 \text{ ksi} \end{cases}$$

**Design Compressive Strength**

(Eqn. E3-1)

$$P_n = F_{cr} * A = 712.0 \text{ kips}$$

$$\phi_c = 0.90$$

$$\phi_c P_n = \phi_c P_n = 640.8 \text{ kips}$$

**Lateral Torsional Buckling (LTB)**

The limiting lengths $L_p$ and $L_r$ are determined according to the AISC Spec. Eqns. F2-5 and F2-6, as follows:

$$L_p = 1.76 * r_y * \sqrt{\frac{E}{F_y}} / 12 = 8.69 \text{ ft}$$

$$L_{r1} = \sqrt{\frac{J * 1.0}{S_x * h_o}} = 0.05$$

$$L_{r2} = \sqrt{1 + \frac{6.76 \left( \frac{0.7 * F_y * S_x * h_o}{E * J * 1.0} \right)^2}{E}} = 1.56$$

$$L_r = 1.95/12 * r_{ts} * \frac{E}{0.7 * F_y} * L_{r1} * L_{r2} = 29.41 \text{ ft}$$

Case = IF($L_b > L_r$;"ELTB";IF($L_b \leq L_p$; "No LTB";"InLTB")) = InLTB

("ELTB" refers to elastic LTB, and "InLTB" refers to Inelastic LTB.)

According to the AISC Spec. Eqn. F2-2:

$$M_1 = \text{MIN}(M_{px}; C_b * (M_{px} - (M_{px} - 0.7 * 1/12 * F_y * S_x) * (L_b - L_p)) / (L_r - L_p)) = 433 \text{ kip*ft}$$

According to the AISC Spec. Eqn. F2-4:

$$F_{cr} = C_b * \frac{E}{\left( \frac{L_b + 0.01}{12/r_{ts}} \right)^2} = 79.4 \text{ ksi}$$

$$F_{cr,mod} = \sqrt{1 + 0.078 * J * \frac{1.0}{S_x * h_o} * \left( \frac{L_b * 12}{r_{ts}} \right)^2} = 1.3 \text{ ksi}$$

According to the AISC Spec. Eqns. F2-3:

$$M_2 = \text{MIN}(M_{px}; F_{cr} * S_x / 12 * F_{cr,mod}) = 479 \text{ kip*ft}$$
According to the AISC Spec. Eqn. F2-2:

\[ M_{nx2} = \text{IF (Case="No L.T.B.",M_p,M_{nx1},M_{nx2})} = 433 \text{ kip*ft} \]

**Design Flexure Moments in Major/Minor Axes**

\[ \phi_b = 0.90 \]

\[ M_{nx} = \text{MIN}(M_{px},M_{nx1},M_{nx2}) = 433 \text{ kip*ft} \]

\[ M_{ny} = \text{MIN}(M_{py},M_{ny1}) = 154 \text{ kip*ft} \]

**Calculate the Available Flexural and Axial Strengths**

\[ F_{ca} = \frac{\phi_c * P_n}{A} = 32.04 \text{ ksi} \]

\[ F_{bcx} = 12* \frac{\phi_b * M_{nx}}{S_x} = 45.40 \text{ ksi} \]

\[ F_{bcy} = 12* \frac{\phi_b * M_{ny}}{S_y} = 68.73 \text{ ksi} \]

**Calculate the Actual Flexural and Axial Stresses**

\[ f_{ra} = \frac{P_u}{A} = 10.00 \text{ ksi} \]

\[ f_{rbx} = 12* \frac{M_{ux}}{S_x} = 5.83 \text{ ksi} \]

\[ f_{rby} = 12* \frac{M_{uy}}{S_y} = 24.79 \text{ ksi} \]

**Check the Combined Stress Ratio**

\[ \text{(AISC Specification Section H2)} \]

\[ \text{Stress}_\text{ratio} = \frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{bcx}} + \frac{f_{rby}}{F_{bcy}} = 0.80 \]

\[ \text{Safety} = \text{IF}(\text{Stress}_\text{ratio} < 1, "Safe", "Unsafe") = \text{Safe} \]
Design Summary

\[ f_{ra} = \frac{P_u}{A} = 10.0 \text{ ksi} \]

\[ F_{ca} = \frac{\phi_c * P_n}{A} = 32.0 \text{ ksi} \]

\[ f_{rbx} = 12^* \frac{M_{ux}}{S_x} = 5.8 \text{ ksi} \]

\[ F_{bcx} = 12^* \frac{\phi_b * M_{nx}}{S_x} = 45.4 \text{ ksi} \]

\[ f_{rby} = 12^* \frac{M_{uy}}{S_y} = 24.8 \text{ ksi} \]

\[ F_{bcy} = 12^* \frac{\phi_b * M_{ny}}{S_y} = 68.7 \text{ ksi} \]

\[ \text{Stress\_ratio} = \frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{bcx}} + \frac{f_{rby}}{F_{bcy}} = 0.80 \]

\[ \text{Safety} = \text{IF(Stress\_ratio<1;"Safe","Unsafe")} = \text{Safe} \]
Design of W-Shapes Subjected to Axial Compression and Moments including the Second Order Effect

Materials
Grade: SEL("Material/ASTM"; NAME; ) = A992
\( F_y = \) TAB("Material/ASTM";F\_y;NAME=Grade) = 50 ksi
\( E = 29000 \) ksi

Beam Length and \( C_b \) (The member is not subjected to sidesway)
Unsupported length, \( L_b = 14.00 \) ft
Strong axis unbraced length, \( kL_{in} = 14.00 \) ft
Weak axis and torsional unbraced length, \( kL_{out} = 14.00 \) ft
From Table 3-1 (AISC ), \( C_b = 1.14 \)

Given Straining Actions (Not including second-order effects)
\( P_D = 5.0 \) kips
\( P_L = 15.0 \) kips
\( M_{xD} = 15.0 \) kip*ft
\( M_{xL} = 45.0 \) kip*ft
\( M_{yD} = 2.0 \) kip*ft
\( M_{yL} = 6.0 \) kip*ft
From Chapter 2 of ASCE/SEI 7, the required strength (not considering second-order effects) is:
\( P_u = 1.2*P_D + 1.6*P_L = 30.0 \) kips
\( M_{ux1} = 1.2*M_{xD} + 1.6*M_{xL} = 90.0 \) kip*ft
\( M_{uy1} = 1.2*M_{yD} + 1.6*M_{yL} = 12.0 \) kip*ft

Interactive Design Aids for Structural Engineers
Section Details

sec.: SEL("AISC/W";NAME; ) = W10X33

depth, d= TAB("AISC/W";d;NAME=sec.) = 9.73 in

Web th., t_w= TAB("AISC/W";t_w;NAME=sec.) = 0.29 in

Flange width, b_f= TAB("AISC/W";b_f;NAME=sec.) = 7.96 in

Flange th., t_f= TAB("AISC/W";t_f;NAME=sec.) = 0.44 in

Gross Area, A= TAB("AISC/W";A;NAME=sec.) = 9.71 in²

I_x= TAB("AISC/W";I_x;NAME=sec.) = 171.00 in⁴

I_y= TAB("AISC/W";I_y;NAME=sec.) = 36.60 in⁴

(I_x and I_y are the moment of inertia about x- and y-axes, respectively)

Plastic sec. modulus, Z_x= TAB("AISC/W";Z_x;NAME=sec.) = 38.80 in³

Elastic sec. modulus, S_x= TAB("AISC/W";S_x;NAME=sec.) = 35.00 in³

Plastic sec. modulus, Z_y= TAB("AISC/W";Z_y;NAME=sec.) = 14.00 in³

Elastic sec. modulus, S_y= TAB("AISC/W";S_y;NAME=sec.) = 9.20 in³

r_x= TAB("AISC/W";r_x;NAME=sec.) = 4.19 in

r_y= TAB("AISC/W";r_y;NAME=sec.) = 1.94 in

(r_x and r_y are the radius of gyration about x- and y- axis, respectively)

Torsional constant, J= TAB("AISC/W";J;NAME=sec.) = 0.58 in⁴

r_ts= TAB("AISC/W";r_ts;NAME=sec.) = 2.20 in

h_o= TAB("AISC/W";h_o;NAME=sec.) = 9.30 in

(r_ts is the Effective radius of gyration for the L.T.B. and h_o is distance between C.L. of flanges)

AISC Specification Eqn. (F2-1), yielding Moment in major axis (M_px):

M_px = Z_x * F_y * 1/12 = 162 kip*ft

AISC Specification Eqn. (F6-1), yielding Moment in minor axis (M_py):

M_py = MIN (Z_y * F_y * 1/12; 1.6/12 * S_y * F_y) = 58 kip*ft
Chapter 3: Steel Design
W-Shape Subjected to P and M including the Second Order Effect

Required Flexural Strength (including second-order amplification)

Use the approximate method of second-order analysis procedure from AISC Specification Appendix 8. Because the member is not subject to sidesway, only P-δ amplifiers need to be added.

\[ B_1 = \frac{C_m}{1 - \alpha \cdot P_r / P_{e1}} \geq 1 \quad \text{(Spec. Eq. A-8-3)} \]

The x-x axis flexural magnifier is,

\[ C_{mx} = 1.00 \]

\[ P_{e1} = \frac{\frac{2}{\pi} \cdot E \cdot I_x}{(k L_{in} \cdot 12)^2} = 1734 \text{ kips} \]

\[ \alpha = 1.00 \]

\[ B_{1x} = \frac{C_{mx}}{1 - \alpha \cdot P_u / P_{e1}} = 1.02 \]

\[ M_{ux1} = B_{1x} \cdot M_{ux1} = 91.8 \text{ kip*ft} \]

The Y-Y axis flexural magnifier is,

\[ P_{e2} = \frac{\frac{2}{\pi} \cdot E \cdot I_y}{(k L_{out} \cdot 12)^2} = 371.2 \text{ kips} \]

\[ C_{my} = 1.00 \]

\[ B_{1y} = \frac{C_{my}}{1 - \alpha \cdot P_u / P_{e2}} = 1.09 \]

\[ M_{uy1} = B_{1y} \cdot M_{uy1} = 13.1 \text{ kip*ft} \]

Element Classification

(1) Web:

\[ h/t_w, \lambda_w = \text{TAB("AISC/W";h/t_w;NAME=sec.)} = 27.10 \]

According to AISC Specification Table B4.1 Case 9, the limiting width-to-thickness ratio for the web is:

\[ \text{Web Class= IF(} \lambda_w \leq 3.76 \cdot \sqrt{E/F_y}, \text{"Compact"; "Non-Compact") = Compact} \]

(2) Comp. flange:

\[ b_f/2t_f, \lambda_{rf} = \text{TAB("AISC/W";b_f/2t_f;NAME=sec.)} = 9.15 \]

According to AISC Specification Table B4.1 Case 1, the limiting width-to-thickness ratios for the compression flange are:

\[ \lambda_{pf} = 0.38 \cdot \sqrt{E/F_y} = 9 \]

\[ \lambda_{rf} = 1.00 \cdot \sqrt{E/F_y} = 24 \]

\[ \text{Fl Class= IF}(\lambda_f \leq \lambda_{pf}, \text{"Compact";} \text{IF}(\lambda_f > \lambda_{rf}, \text{"Slender";} \text{"Non-Compact"}) = \text{Non-Compact} \]
Chapter 3: Steel Design

W-Shape Subjected to P and M including the Second Order Effect

The available strength provided by AISC Specification Sections F3.1, F3.2, F6.1 and F6.2, the nominal flexural moments in strong/weak axes are calculated as follows, satisfying the condition of compression Flange Local Buckling:

\[
M_{nx1a} = M_{px} - 0.7F_y S_x * \frac{\lambda_f}{12} = 60 \text{ kip*ft}
\]

\[
M_{nx1} = \text{IF(Fl Class="Compact";} M_{px}; \text{IF(Fl Class="Compact";} M_{py}; \frac{\lambda_f}{12} - \frac{\lambda_{pf}}{12})) = 161 \text{ kip*ft}
\]

\[
M_{ny1a} = M_{py} - 0.7F_y S_y * \frac{\lambda_f}{12} = 31 \text{ kip*ft}
\]

\[
M_{ny1} = \text{IF(Fl Class="Compact";} M_{py}; \text{IF(Fl Class="Compact";} M_{py}; \frac{\lambda_f}{12} - \frac{\lambda_{pf}}{12})) = 58 \text{ kip*ft}
\]

Slenderness Check (According to section E2)

For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

\[
\lambda_x = \frac{kL_{in}}{r_x} * 12 = 40.1
\]

\[
\lambda_y = \frac{kL_{out}}{r_y} * 12 = 86.6
\]

Then, the governed slenderness (\(\lambda_{max}\)):

\[
\lambda_{max} = \text{MAX}(\lambda_x; \lambda_y) = 86.6
\]

Critical Stresses

The available critical stresses may be interpolated from AISC Manual Table 4-22 or calculated directly as follows:

- Calculate the elastic critical buckling stress, \(F_e\):

\[
F_e = \frac{2\pi E}{\lambda_{max}^2} = 38.2 \text{ ksi}
\]

- Calculate the flexural buckling stress, \(F_{cr}\) (Eqns. E3-2 and E3-3):

\[
\lambda_1 = 4.71*\sqrt{\frac{E}{F_y}} = 113
\]

\[
F_{cr} = \text{IF}(\lambda_{max} \leq \lambda_1; \ (0.658 \frac{F_y}{F_e})^\lambda_{max} * F_y; 0.877*F_e) = 28.9 \text{ ksi}
\]
### Design Compressive Strength (Eqn. E3-1)

\[
P_n = \Phi_c \cdot F_{cr} \cdot A = 281 \text{ kips}
\]

\[
\Phi_c = 0.90
\]

\[
\Phi_c P_n = \Phi_c \cdot P_n = 253 \text{ kips}
\]

### Lateral Torsional Buckling (LTB)

The limiting lengths \(L_p\) and \(L_r\) are determined according to the AISC Spec. Eqns. F2-5 and F2-6, as follows:

\[
L_p = 1.76 \cdot r_y \cdot \sqrt{E/F_y}/12 = 6.85 \text{ ft}
\]

\[
L_{r1} = \frac{J \cdot 1.0}{S_x \cdot h_o} = 0.04
\]

\[
L_{r2} = \sqrt{1 + \left(6.76 \cdot \left(\frac{0.7 \cdot F_y \cdot S_x \cdot h_o}{E \cdot J \cdot 1.0}\right)^2 \right)} = 1.66
\]

\[
L_r = 1.95/12 \cdot r_{ts} \cdot \frac{E}{0.7 \cdot F_y} \cdot \sqrt{L_{r1} \cdot L_{r2}} = 19.67 \text{ ft}
\]

Case = IF \((L_b > L_r; "ELTB"; IF(L_b \leq L_p; "No LTB"; "InLTB")) = \) InLTB

("ELTB" refers to elastic LTB and "InLTB" refers to inelastic LTB)

According to the AISC Spec. Eqn. F2-2:

\[
M_{1a} = M_{px} \cdot 0.7 \cdot 1/12 \cdot F_Y \cdot S_x = 59.9 \text{ kip*ft}
\]

\[
M_1 = \text{MIN}(M_{px}; C_b \cdot (M_{px} - M_{1a} \cdot (L_b - L_p)/(L_r - L_p))) = 147 \text{ kip*ft}
\]

According to the AISC Spec. Eqn. F2-4:

\[
F_{cr} = C_b \cdot \pi^2 \cdot E / ((L_b + 0.01) \cdot 12/r_{ts})^2 = 55.87 \text{ ksi}
\]

\[
F_{cr,mod} = \sqrt{(1 + 0.078 \cdot J \cdot 1.0/(S_x \cdot h_o)) \cdot (L_b + 12/r_{ts})^2} = 1.35
\]

According to the AISC Spec. Eqns. F2-3:

\[
M_2 = \text{MIN}(M_{px} \cdot F_{cr} \cdot S_x/12 \cdot F_{cr,mod}) = 162 \text{ kip*ft}
\]

According to the AISC Spec. Eqn. F2-2:

\[
M_{nx2} = \text{IF} \ (\text{Case}="\text{No LTB}"; M_{px}; \text{IF} \ (\text{Case}="\text{InLTB}"; M_1; M_2)) = 147.0 \text{ kip*ft}
\]

### Design Flexure Moment in Major/Minor Axes

\[
\Phi_b = 0.90
\]

\[
M_{nx} = \text{MIN}(M_{px} \cdot M_{nx1} \cdot M_{nx2}) = 147.0 \text{ kip*ft}
\]

\[
M_{ny} = \text{MIN}(M_{py} \cdot M_{ny1}) = 58.0 \text{ kip*ft}
\]
Calculate The Available Flexural and Axial Strengths

\[ P_c = \Phi_c \cdot P_n = 252.9 \text{ kips} \]
\[ M_{cx} = \Phi_b \cdot M_{nx} = 132.3 \text{ kip*ft} \]
\[ M_{cy} = \Phi_b \cdot M_{ny} = 52.2 \text{ kip*ft} \]

Check The Combined Stress Ratio (AISC Specification Section H1-1a and H1-1b)

Axial ratio, \( p = \frac{P_u}{P_c} = 0.12 \)

Moments ratio, \( m = \frac{M_{ux}}{M_{cx}} + \frac{M_{uy}}{M_{cy}} = 0.94 \)

Stress\_ratio = IF\( (p \geq 0.2; \ (p+8/9*m);(p/2+m)) = 1.00 \text{ in} \)
Safety = IF\( (\text{Stress\_ratio} \leq 1; \text{"Safe"};\text{"Unsafe"}) = \text{Safe} \)

Design Summary

\[ P_c = \Phi_c \cdot P_n = 252.9 \text{ kips} \]
\[ M_{cx} = \Phi_b \cdot M_{nx} = 132.3 \text{ kip*ft} \]
\[ M_{cy} = \Phi_b \cdot M_{ny} = 52.2 \text{ kip*ft} \]

Stress\_ratio = IF\( (p \geq 0.2; \ (p+8/9*m);(p/2+m)) = 1.00 \text{ in} \)
Safety = IF\( (\text{Stress\_ratio} \leq 1; \text{"Safe"};\text{"Unsafe"}) = \text{Safe} \)
Design of HSS-Shapes Subjected to Moment about Strong Axis

Materials

Grade: SEL("Material/ASTM";NAME; ) = A500

Fy= TAB("Material/ASTM";F_y;NAME=Grade) = 46 ksi

E= 29000 ksi

Beam Length and C_b

Total length, L= 50.00 ft

Design Moments and Uniform Live Load

Ultimate moment, M_u= 150.00 kip*ft

Ultimate moment due to live load case, M_L= 80.00 kip*ft

Ultimate Shear force, Q_u= 66 kips

Section Details

sec.: SEL("AISC/HSS";NAME; ) = HSS18X6X3/8

depth, h_t= TAB("AISC/HSS";H_t;NAME=sec.) = 18.00 in

HSS th., t_des= TAB("AISC/HSS";t_des;NAME=sec.) = 0.349 in

HSS width, b= TAB("AISC/HSS";B;NAME=sec.) = 6.00 in

Plastic sec. modulus, Z_x= TAB("AISC/HSS";Z_x;NAME=sec.) = 86.40 in^3

Elastic sec. modulus, S_x= TAB("AISC/HSS";S_x;NAME=sec.) = 66.90 in^3

Inertia about x-axis, I_x= TAB("AISC/HSS";I_x;NAME=sec.) = 602.00 in^4

Yielding Moment, M_p= Z_x*F_y*1/12 = 331 kip*ft
Element Classification

(1) Web:
\[ h/t_{des}, \lambda_w = \text{TAB("AISC/HSS";h/t_{des};NAME=sec.)} = 48.70 \]
Determine the limiting ratio for a compact HSS web in flexure from AISC Specification Table B4.1b Case 19.
\[ \lambda_{wr} = 2.42 \sqrt{(E/F_y)} = 60.8 \]
\[ \text{Web Class} = \text{IF}(\lambda_w \leq \lambda_{wr}; "Compact"; "Non-Compact") = \text{Compact} \]

(2) Comp. flange:
\[ b/t_{des}, \lambda_f = \text{TAB("AISC/HSS";b/t_{des};NAME=sec.)} = 14.20 \]
Determine the limiting ratio for a slender HSS flange in flexure from AISC Specification Table B4.1b Case 17.
\[ \lambda_{fr} = 1.12 \sqrt{(E/F_y)} = 28.1 \]
\[ \text{Fl Class} = \text{IF}(\lambda_f \leq \lambda_{fr}; "Compact"; "Non-Compact") = \text{Compact} \]
\[ M_{n1a} = M_p - F_y \cdot S_x * 1/12 = 74.55 \text{ kip*ft} \]
\[ \lambda_{fa} = 3.57 \lambda_f \sqrt{(F_y/E)} = -1.98 \]
\[ M_{n1b} = \text{MIN}(M_p; M_p - M_{n1a} \cdot \lambda_{fa}) = 331.0 \text{ kip*ft} \]
\[ M_{n1} = \text{IF}(\text{Fl Class}="Compact"; M_p; M_{n1b}) = 331.0 \text{ kip*ft} \]
(Note that For HSS with noncompact flanges and compact webs, AISC Specification Section F7.2(b) applies)

Check The Available Flexure Strength
\[ \phi M_n = 0.90 \cdot \text{MIN}(M_p; M_{n1}) = 298 \text{ kip*ft} \]
\[ \text{Safety} = \text{IF}(\phi M_n \geq M_u; "Safe"; "Unsafe") = \text{Safe} \]
\[ \text{Moment ratio} = M_u/\phi M_n = 0.50 \]

Check Shear Strength
From AISC Specification Section G5, if the exact radius is unknown, \( h \) shall be taken as the corresponding outside dimension minus three times the design thickness.
\[ h = H_t - 3 \cdot t_{des} = 17 \text{ in} \]
\[ \lambda_w = \text{TAB("AISC/HSS";h/t_{des};NAME=sec.)} = 48.70 \]
For rectangular HSS in shear, use AISC Specification Section G2.1 with \( A_w = 2ht \) (per AISC Specification Section G5) and \( k_v = 5 \).

\[
\begin{align*}
\lambda_{w1} &= 1.1\sqrt{(k_v E/F_y)} = 62 \\
\lambda_{w2} &= 1.37\sqrt{(k_v E/F_y)} = 77 \\
C_{va} &= 1.51*5E/(F_y \lambda_{w}^2) = 2.0 \\
C_v &= IF(\lambda_w \leq \lambda_{w1}; 1; IF((\lambda_w > \lambda_{w1} AND \lambda_w \leq \lambda_{w2}); \lambda_{w1}/\lambda_w; C_{va})) = 1 \\
A_w &= 2*h*t_{des} = 12 \text{ in}^2 \\
\end{align*}
\]

Nominal shear strength (\( V_n \)):
\[
V_n = 0.6*F_y A_w * C_v = 331.20 \text{ kips}
\]

From AISC Specification Section G1, the available shear strength is:
\[
\Phi_v = 0.90 \\
\Phi_v V_n = \Phi_v * V_n = 298.08 \text{ kips}
\]

Shear safety:
\[
\text{IF}(\Phi_v * V_n > Q_u; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}
\]

Check Deflection
\[
\Delta_{all} = \frac{L}{240} = 0.21 \text{ ft}
\]

\[
W_{eq \ (LL)}, W_L = \frac{8 * M_L}{L^2} = 0.26 \text{ kip/ft}
\]

\[
\Delta_{act} = \frac{5 * W_L * L^4}{384 * E * I_x} = 0.175 \text{ ft}
\]

Deflection safety, \( D_s = \text{IF}(\Delta_{all} > \Delta_{act}; \text{"Safe"}; \text{"Increase section"}) = \text{Safe} \)

Design Summary
\[
\Phi M_n = 0.90*\text{MIN}(M_p; M_{n1}) = 298 \text{ kip*ft}
\]

Safety:
\[
\text{IF}(\Phi M_n > M_u; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}
\]

Moment ratio:
\[
M_u/\Phi M_n = 0.50
\]

\[
\Phi_v V_n = \Phi_v * V_n = 298 \text{ kips}
\]

Shear safety:
\[
\text{IF}(\Phi_v * V_n > Q_u; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}
\]

\[
\Delta_{act} = \frac{5 * W_L * L^4}{384 * E * I_x} = 0.00 \text{ ft}
\]

\[
\Delta_{all} = \frac{L}{240} = 0.21 \text{ ft}
\]

Deflection safety, \( D_s = \text{IF}(\Delta_{all} > \Delta_{act}; \text{"Safe"}; \text{"Increase section"}) = \text{Safe} \)
Design of W-Shapes Subjected to Tension Force in a Bolted Connection

Materials

Grade: SEL("Material/ASTM"; NAME; ) = A992

\( F_y = \) TAB("Material/ASTM";F_y;NAME=Grade) = 50 ksi

\( F_u = \) TAB("Material/ASTM";F_u;NAME=Grade) = 65 ksi

Buckling Lengths

Member length, \( L = 25.00 \) ft

Axial Loads

Axial dead load, \( P_D = 30 \) kips

Axial Live load, \( P_L = 90 \) kips

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

Ultimate load, \( P_u = 1.2*P_D + 1.6*P_L = 180 \) kips

Section and Connection Details

\( \text{sec.:} = \text{SEL("AISC/W";NAME; ) = W12X106} \)

depth, \( d_0 = \) TAB("AISC/W";d;NAME=sec.) = 12.90 in

Web th., \( t_w = \) TAB("AISC/W";t_w;NAME=sec.) = 0.61 in

Flange width, \( b_f = \) TAB("AISC/W";b_f;NAME=sec.) = 12.20 in

Flange th., \( t_f = \) TAB("AISC/W";t_f;NAME=sec.) = 0.99 in

Gross Area, \( A_g = \) TAB("AISC/W";A;NAME=sec.) = 31.20 in\(^2\)

\( r_x = \) TAB("AISC/W";r_x;NAME=sec.) = 5.47 in

\( r_y = \) TAB("AISC/W";r_y;NAME=sec.) = 3.11 in

\( (r_x \text{ and } r_y \text{ are the radius of gyration about } x- \text{ and } y- \text{ axis}) \)
Chapter 3: Steel Design

W-Shape Subjected to Tension Force in a Bolted Connection

Rounded depth, \( d_{WT} \):
\[
TAB("AISC/W";d_{WT};NAME=sec.) = 13.00 \text{ in}
\]

\( d_{WT} \):
\[
TAB("AISC/WT";NAME;d_{ro}=d_{WT}/2) = WT6X53
\]

\( y \):
\[
TAB("AISC/WT";y;NAME=WT) = 1.190 \text{ in}
\]

Bolt diameter:
\[
SEL("AISC/Bolt";Size;) = d_{3/4}
\]

\( d_b \):
\[
TAB("AISC/Bolt";dia;Size=Bolt\_diameter) = 0.750 \text{ in}
\]

Hole diameter, \( d_h \):
\[
d_b + 1/16 = 0.813 \text{ in}
\]

Connection length, \( l \):
\[
9.00 \text{ in}
\]

Check Tensile Yielding

From AISC Manual Table 5-1, the tensile yielding strength is:
\[
\phi_{t1} = 0.90
\]
\[
P_{n1} = \phi_{t1} \cdot F_y \cdot A_g = 1404.0 \text{ kips}
\]

Yield\_safety = \( \text{IF}(P_u \leq P_{n1} ; "\text{Safe}";"\text{Unsafe}"") \)

Safe

Check Tensile Rupture

Calculate the shear lag factor, \( U \), as the larger of the values from AISC specification section D3, Table D3.1 case 2 and case 7. From AISC Specification Section D3, for open cross sections, \( U \) need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

Case 2:
\[
U_1 = \frac{2 \cdot b_t \cdot t_f}{A_g} = 0.774
\]

Case 2: Check as two WT-shapes per AISC Specification Commentary Figure C-D3.1

\[
U_2 = \frac{y}{l} = 0.868
\]

Case 7:
\[
U_3 = \text{IF}(b_t \geq 2/3 \cdot d_o; 0.90; 0.85) = 0.900
\]

\( U = \text{MAX}(U_1;U_2;U_3) = 0.900 \)

Effective Net Area

Calculate \( A_n \) using AISC Specification Section B4.3.
\[
A_n = A_g - 4 \times (d_h + 1/16) \times t_f = 27.73 \text{ in}^2
\]

Calculate \( A_g \) using AISC Specification Section D3
\[
A_g = A_n \times U = 24.96 \text{ in}^2
\]
Available Tensile Rupture Strength

\[ P_2 = F_u \times A_e = 1622.4 \text{ kips} \]

\[ \phi_{t2} = 0.75 \]

\[ P_{n2} = P_2 \times \phi_{t2} = 1216.8 \text{ kips} \]

\[ \text{Rupture\_safety} = \text{IF}(P_u \leq P_{n1}; "Safe"; "Unsafe") = \text{Safe} \]

Slenderness Check (According to section D1)

For members designed on the basis of compression, the slenderness ratio KL/r should not exceed 300.

\[ \lambda_{\text{max}} = \frac{L}{r_y} \times 12 = 96.5 \]

\[ \text{Slenderness\_limit} = \text{IF}(\lambda_{\text{max}} \leq 300; "Safe"; "Unsafe") = \text{Safe} \]

Design Summary

Ultimate load, \( P_u = 1.2 \times P_D + 1.6 \times P_L \) = 180.0 kips

\[ P_{n1} = \phi_{t1} \times F_y \times A_g = 1404.0 \text{ kips} \]

\[ \text{Yield\_safety} = \text{IF}(P_u \leq P_{n1}; "Safe"; "Unsafe") = \text{Safe} \]

\[ P_{n2} = P_2 \times \phi_{t2} = 1216.8 \text{ kips} \]

\[ \text{Rupture\_safety} = \text{IF}(P_u \leq P_{n1}; "Safe"; "Unsafe") = \text{Safe} \]

\[ \text{Slenderness\_limit} = \text{IF}(\lambda_{\text{max}} \leq 300; "Safe"; "Unsafe") = \text{Safe} \]
Design of WT-Shapes Subjected to Tension Force in Welded Connections

Materials

Grade: SEL("Material/ASTM"; NAME; ) = A992

\[ F_y = \text{TAB}("Material/ASTM";F_y;NAME=Grade) = 50 \text{ ksi} \]

\[ F_u = \text{TAB}("Material/ASTM";F_u;NAME=Grade) = 65 \text{ ksi} \]

Buckling Length

Member length, \( L = 30.00 \text{ ft} \)

Axial Loads

Axial dead load, \( P_D = 40 \text{ kips} \)

Axial Live load, \( P_L = 120 \text{ kips} \)

From Chapter 2 of ASCE/SEI 7, the required compressive strength is:

\[ P_u = 1.2P_D + 1.6P_L = 240 \text{ kips} \]

Section and Connection Details

sec.: SEL("AISC/WT";NAME; ) = WT6X20

\[ d_o = \text{TAB}("AISC/WT";d;NAME=sec.) = 5.97 \text{ in} \]

\[ t_w = \text{TAB}("AISC/WT";t_w;NAME=sec.) = 0.29 \text{ in} \]

\[ b_f = \text{TAB}("AISC/WT";b_f;NAME=sec.) = 8.01 \text{ in} \]

\[ t_f = \text{TAB}("AISC/WT";t_f;NAME=sec.) = 0.515 \text{ in} \]

\[ A_g = \text{TAB}("AISC/WT";A;NAME=sec.) = 5.84 \text{ in}^2 \]

\[ r_x = \text{TAB}("AISC/WT";r_x;NAME=sec.) = 1.57 \text{ in} \]

\[ r_y = \text{TAB}("AISC/WT";r_y;NAME=sec.) = 1.94 \text{ in} \]

\( (r_x \text{ and } r_y \text{ are the radius of gyration about } x- \text{ and } y- \text{ axis}) \)

Distance to centroid, \( y = \text{TAB}("AISC/WT"; y;NAME=sec.) = 1.09 \text{ in} \)

Connection length, \( l = 16.00 \text{ in} \)
Check Tensile Yielding

From AISC Manual Table 5-1, the tensile yielding strength is:

$$\Phi_{t1} = 0.90$$

$$P_{n1} = \Phi_{t1} \cdot F_y \cdot A_g = 262.8 \text{ kips}$$

$$\text{Yield}_{\text{safety}} = \text{IF}(P_u \leq P_{n1}; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}$$

Check Tensile Rupture

Calculate the shear lag factor, $U$, as the larger of the values from AISC Specification Section D3, Table D3.1 case 2 and case 7. From AISC Specification Section D3, for open cross sections, $U$ need not be less than the ratio of the gross area of the connected element(s) to the member gross area.

$$U_1 = \frac{b_f \cdot t_f}{A_g} = 0.706$$

Case 2: Check as two WT-shapes per AISC Specification Commentary Figure C-D3.1

$$U_2 = 1 \cdot \frac{y}{l} = 0.932$$

Case 7:

$$U_3 = \text{IF}(b_f \geq 2/3 \cdot d_o; 0.90; 0.85) = 0.900$$

$$U = \text{MAX}(U_1; U_2; U_3) = 0.932$$

Effective Net Area

Calculate $A_n$ using AISC Specification Section B4.3.

$$A_n = A_g = 5.8 \text{ in}^2$$

Calculate $A_e$ using AISC Specification Section D3

$$A_e = A_n \cdot U = 5.4 \text{ in}^2$$

Available Tensile Rupture Strength

$$P_2 = F_u \cdot A_e = 351.0 \text{ kips}$$

$$\Phi_{t2} = 0.75$$

$$P_{n2} = P_2 \cdot \Phi_{t2} = 263.3 \text{ kips}$$

$$\text{Rupture}_{\text{safety}} = \text{IF}(P_u \leq P_{n1}; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}$$
Chapter 3: Steel Design
WT-Shape Subjected to Tension Force in Welded Connections

Slenderness Check (According to section D1)

For members designed on the basis of compression, the slenderness ratio \( KL/r \) should not exceed 300.

\[
\begin{align*}
\lambda_{\text{max}} &= \frac{L}{r_{\text{min}}} \\
\lambda_{\text{max}} &= \frac{L}{r_{\text{min}}} \times 12 \quad = \quad 229.3 \\
\text{Slenderness limit} &= \text{IF}(\lambda_{\text{max}} \leq 300; \text{"Safe"}; \text{"Unsafe"}) \quad = \quad \text{Safe}
\end{align*}
\]

Design Summary

\[
\begin{align*}
\text{Ultimate load, } P_u &= 1.2P_D + 1.6P_L \quad = \quad 240.0 \text{ kips} \\
\Phi_{n1} &= \Phi_{t1} \times F_y \times A_g \quad = \quad 262.8 \text{ kips} \\
\text{Yield safety} &= \text{IF}(P_u \leq P_{n1}; \text{"Safe"}; \text{"Unsafe"}) \quad = \quad \text{Safe} \\
\Phi_{n2} &= \Phi_{t2} \times P_2 \quad = \quad 263.3 \text{ kips} \\
\text{Rupture safety} &= \text{IF}(P_u \leq P_{n1}; \text{"Safe"}; \text{"Unsafe"}) \quad = \quad \text{Safe} \\
\text{Slenderness limit} &= \text{IF}(\lambda_{\text{max}} \leq 300; \text{"Safe"}; \text{"Unsafe"}) \quad = \quad \text{Safe}
\end{align*}
\]
Chapter 3: Steel Design
Interior Panel of Built-Up Girder with Transverse Stiffeners

Design of Interior Panel of Built-Up Girder with Transverse Stiffeners Subjected to Shear Force

**Materials**

- Grade: SEL("Material/ASTM"; NAME; ) = A36
- \(F_y\) = TAB("Material/ASTM";F_y;NAME=Grade) = 36 ksi
- E= 29000 ksi

**Loads**

The required shear strength at the start of this panel from the end, \(V_u\):

- \(V_u\) = 120 kips

**Section Details**

- Depth, \(d\) = 36 in
- Web thickness, \(t_w\) = 0.3125 in
- Clear distance between stiffeners, \(a\) = 99 in
- Upper flange width, \(b_{fc}\) = 12 in
- Upper flange thickness, \(t_{fc}\) = 1.50 in
- Lower flange width, \(b_{ft}\) = 12.00 in
- Lower flange thickness, \(t_{ft}\) = 1.50 in

**Shear Strength for this Panel**

\[
\begin{align*}
h &= d - t_{fc} - t_{ft} = 33.00 \text{ in} \\
\psi &= \frac{a}{h} = 3.00 \\
k_{v_1} &= 5 + \left(\frac{5}{\psi^2}\right) = 5.56 \\
\end{align*}
\]

Based on AISC Specification Section G2.1, \(k_v=5\) when \(a/h>3\) or \(a/h>\sqrt[2]{\frac{260(h/tw))^2}

\[
\begin{align*}
\lambda_w &= \frac{h}{tw} = 106 \\
\psi_1 &= \left(\frac{260}{\lambda_w}\right)^2 = 6.02 \\
Use \ k_v &= \text{IF}(\psi>3 \text{ AND } \psi>\psi_1;5; k_{v_1}) = 5.56 \\
\lambda_{w1} &= 1.10\sqrt{(k_v\times E/F_y)} = 74 \\
\lambda_{w2} &= 1.37\sqrt{(k_v\times E/F_y)} = 92
\end{align*}
\]

Interactive Design Aids for Structural Engineers
Chapter 3: Steel Design
Interior Panel of Built-Up Girder with Transverse Stiffeners

Calculate \( C_v \) according to Eqns. G2-2, G2-3, G2-4 and G2-5:

\[
C_{v1} = \begin{cases} 
1 & \text{if } \lambda_w \leq \lambda_{w1}; \text{IF}(\lambda_w > \lambda_{w1}, \lambda_w \leq \lambda_{w2}; \lambda_w < \lambda_{w1}, 1.51 * k_v * E/(F_y * \lambda_w^2)) \leq 0.60 
\end{cases}
\]

Check the additional limits from AISC Specification Section G3.1 for the use of tension field action:

\[
A_w = \frac{d^*t_w}{A_w*2} = 11.3 \text{ in}^2
\]

\[
\eta = \frac{b_{fc} * t_{fc} + b_{ft} * t_{ft}}{b_{fc}} = 0.63
\]

\[
v_{fc} = \frac{h}{b_{fc}} = 2.75
\]

\[
v_{ft} = \frac{h}{b_{ft}} = 2.75
\]

Check_TF = \begin{cases} 
1 & \text{IF} (\psi > \text{MIN}(3; \psi_1) \text{ AND } \eta > 2.5 \text{ AND } v_{fc} > 6 \text{ AND } v_{ft} > 6; "1"; "2") \leq 2 
\end{cases}

Case = \begin{cases} 
P & \text{IF} (\text{Check_TF} = "1"; "NP"; "P") 
\end{cases}

"P": permitted and "NP": not permitted

Calculate \( C_v \) and \( V_n \), according to Section G3-2a and b:

\[
C_{v2} = \begin{cases} 
1 - C_v & \text{IF} (\lambda_w \leq \lambda_{w1}; 1; (C_v + \sqrt{1 - \frac{1}{C_v}})) \leq 0.71 
\end{cases}
\]

Nominal shear strength:

\[
V_n = \begin{cases} 
0.6 * F_y * A_w * C_{v2} & \text{IF} (\text{Case} = "PERMITTED"; 0.6 * F_y * A_w * C_{v2}; 0.6 * F_y * A_w * C_{v1}) \leq 146 \text{ kips} 
\end{cases}
\]

\[
\Phi_v = 0.90
\]

\[
\Phi_v V_n = \Phi_v * V_n \leq 131 \text{ kips}
\]

Shear_safety = \begin{cases} 
\text{Safe} & \text{IF} (\Phi_v * V_n > V_u, "Safe"; "Unsafe") \leq 131 \text{ kips}
\end{cases}

Force_ratio = \frac{V_u}{\Phi_v V_n} = 0.92

Design Summary

\[
V_u = 120 \text{ kips}
\]

\[
\Phi_v V_n = \Phi_v * V_n = 131 \text{ kips}
\]

Force_ratio = \frac{V_u}{\Phi_v V_n} = 0.92

Shear_safety = \begin{cases} 
\text{Safe} & \text{IF} (\Phi_v * V_n > V_u, "Safe"; "Unsafe") \leq 131 \text{ kips}
\end{cases}

Interactive Design Aids for Structural Engineers
Design of End Panel of Built-Up Girder with Transverse Stiffeners Subjected to Shear Force

Materials

- Grade: SEL("Material/ASTM"; NAME; ) = A36
- $F_y = \text{TAB}("Material/ASTM";F_y;NAME=Grade) = 36 \text{ ksi}$
- $E = 29000 \text{ ksi}$

Loads

- Reaction at support, $R_v = 154.0 \text{ kips}$

Section Details

- Depth, $d = 36 \text{ in}$
- Web thickness, $t_w = 0.3125 \text{ in}$
- Clear distance between stiffeners, $a = 60.00 \text{ in}$
- Upper flange width, $b_{fc} = 12 \text{ in}$
- Upper flange thickness, $t_{fc} = 1.50 \text{ in}$
- Lower flange width, $b_{ft} = 12.00 \text{ in}$
- Lower flange thickness, $t_{ft} = 1.50 \text{ in}$

Shear Strength for End Panel

- $h = d - t_{fc} - t_{ft} = 33.00 \text{ in}$
- $\psi = a/h = 1.82$
- $k_{\psi_1} = 5 + (5/\psi^2) = 6.51$

Based on AISC Specification Section G2.1, $k_v=5$ when $a/h>3$ or $a/h>[260/(h/tw)]^2$

- $\lambda_w = h/tw = 106$
- $\psi_1 = (260/\lambda_w)^2 = 6.02$
- Therefore, use $k_v = \text{IF}(\psi>3 \text{ AND } \psi_1>5;5;k_{\psi_1}) = 6.51$
Tension field action is not allowed because the panel is an end panel.

\[
\begin{align*}
\lambda_{w1} &= 1.10 \sqrt{k_v E/F_y} = 80 \\
\lambda_{w2} &= 1.37 \sqrt{k_v E/F_y} = 99
\end{align*}
\]

Calculate \( C_v \) according to Eqns. G2-2, G2-3, G2-4 and G2-5:

\[
C_v = \text{IF}(\lambda_w \leq \lambda_{w1}; 1; \text{IF}(\lambda_w > \lambda_{w1} \text{ AND } \lambda_w \leq \lambda_{w2}; \lambda_{w1}/\lambda_w; 1.51k_v^2 E/(F_y \lambda_w^2))) = 0.705
\]

\[
A_w = d^* t_w = 11.3 \text{ in}^2
\]

Calculate \( V_n \) using Eqn. G2-1:

Nominal shear strength, \( V_n = 0.6F_y A_w C_v \) = 172 kips

\[
\Phi_v = 0.90
\]

\[
\Phi_v V_n = \Phi_v^* V_n = 155 \text{ kips}
\]

Shear _safety = IF(\( \Phi_v^* V_n > R_v \); "Safe"; "Unsafe") = Safe

Force_ratio = \( R_v/\Phi_v V_n \) = 0.99

**Design Summary**

Reaction at support, \( R_v = 154.0 \) kips

\[
\Phi_v V_n = \Phi_v^* V_n = 154.8 \text{ kips}
\]

\[
\text{Force_ratio} = R_v/\Phi_v V_n = 0.99
\]

Interactive Design Aids for Structural Engineers
Design of Composite Beam Subjected to Bending about its Major Axis

Materials
- Grade: SEL("Material/ASTM"; NAME; ) = A992
- $F_y$ = TAB("Material/ASTM"; $F_y$; NAME=Grade) = 50 ksi
- $F_u$ = TAB("Material/ASTM"; $F_u$; NAME=Grade) = 65 ksi
- $E_s$ = 29000 ksi

Beam Length
- Total length, $L$ = 45.00 ft
- Beam spacing, $a$ = 10.0 ft

Concrete Details
- $f'_c$ = 4.0 ksi
- Total thickness, $t_s$ = 7.5 in
- Concrete weight, $\gamma_c$ = 145.0 lb/ft$^3$

Loads
- Superimposed (HVAC, ceiling, floor covering, etc.), $w_{sd}$ = 10.0 lb/ft$^2$
- Live load for construction (temporary loads during concrete placement), $w_{lc}$ = 25.0 lb/ft$^2$
- Live load non-reducible, $w_{ll}$ = 100.0 lb/ft$^2$

Section Details
- sec.: SEL("AISC/W"; NAME; ) = W21X50
- Weight, $w_{sec}$ = TAB("AISC/W"; W; NAME=sec.) = 50.00 lb/ft
- Depth, $d$ = TAB("AISC/W"; d; NAME=sec.) = 20.80 in
- Web th., $t_w$ = TAB("AISC/W"; $t_w$; NAME=sec.) = 0.38 in
- Flange width, $b_f$ = TAB("AISC/W"; $b_f$; NAME=sec.) = 6.53 in
Flange th., \(t_f\) = \(0.54\) in
Gross area, \(A_s\) = \(14.70\) in²
Plastic sec. modulus, \(Z_x\) = \(110.00\) in³
Inertia about x-axis, \(I_x\) = \(984.00\) in⁴

AISC Specification Eqn. (F2-1):
Yielding Moment, \(M_p\) = \(Z_x F_y \times 1/12\) = \(458\) kip*ft

Metal Deck and Stud Connector Details (in accordance with metal deck manufacturer's data)

Rib height, \(h_r\) = \(3.0\) in
Average rib width, \(w_r\) = \(6.0\) in
weight of slab, \(w_s\) = \(75.0\) lb/ft²
Diameter of stud, \(d_{sa}\) = \(0.750\) in
Height of stud (minimum is 4\(d_{sa}\)), \(H_s\) = \(4.50\) in
Extension of the stud above the deck (minimum is 1.5 in), \(H_{sc}\) = \(1.50\) in

Composite Deck and Anchor Requirements
Check composite deck and anchor requirements stipulated in AISC Specification Sections I1.3, I3.2c and I8.

- Conc_strength_check = IF((f'\(_c\) ≤ 10 AND f'\(_c\) ≥ 3); "o.k."; "change f'\(_c\)") = o.k.
- h_r_check = IF(h_r ≤ 3; "o.k."; "decrease h_r") = o.k.
- w_r_check = IF(w_r ≥ 2; "o.k."; "increase w_r") = o.k.
- d_{sa_check} = IF(d_{sa} < 0.75; "o.k."; "decrease d_{sa}") = o.k.
- t_f_check = IF(t_f ≥ d_{sa}/2.5; "o.k."; "decrease d_{sa}") = o.k.
- H_{sc_check} = IF((H_s ≥ (h_r+1.5) AND H_s < t_f-0.5); "o.k."; "unsafe") = o.k.
- H_s_check = IF(H_s ≥ 4*d_{sa}; "o.k."; "increase H_s") = o.k.
- h_c_check = IF((t_f-h_r) ≥ 2; "o.k."; "increase h_c") = o.k.
Design for Pre-Composite Condition

- **Construction Pre-composite Loads:**
  
  \[ w_{D1} = 0.001*(w_s*a + w_{sec}) = 0.80 \text{ kip/ft} \]
  
  \[ w_{L1} = 0.001*(w_{LL}*a) = 0.25 \text{ kip/ft} \]

- **Construction Pre-composite flexural strength, from Chapter 2 of ASCE/SEI 7, the required flexural strength is:**
  
  \[ w_{u1} = 1.2*w_{D1} + 1.6*w_{L1} = 1.36 \text{ kip/ft} \]
  
  \[ M_{u1} = \frac{w_{u1}*L^2}{8} = 344 \text{ kip*ft} \]

Assume that attachment of the deck perpendicular to the beam provides adequate bracing to the compression flange during construction, thus the beam can develop its full plastic moment capacity. The design flexural strength is determined as follows, from AISC Specification Equation F2-1:

\[ \phi_b = 0.90 \]

\[ M_{n1} = \phi_b * M_p = 412 \text{ kip*ft} \]

\[ \text{Flexural_safety1} = \text{IF}(M_{n1} \geq M_{u1}; "Safe"; "Unsafe") = \text{Safe} \]

- **Pre-composite deflection:**
  
  \[ \Delta_{nc} = \frac{5*w_{D1}*(L*12)^4}{384*E_s*I_x} = 2.59 \text{ in} \]
  
  \[ \Delta_{recom} = \frac{L*12}{360} = 1.50 \text{ in} \]

If pre-composite deflection exceeds the recommended limit. One possible solution is to increase the member size. A second solution is to induce camber into the member. So, the user in this step has to determine a solution in case of exceeding the recommended limit.

\[ \text{Camber} = 2.00 \text{ in} \]

\[ \text{deflection_check} = \text{IF}((\Delta_{nc} - \text{Camber}) < \Delta_{recom}; "Safe"; "Unsafe") = \text{Safe} \]

Design for Composite Condition

- **Required Flexural Strength:**
  
  \[ w_{D2} = 0.001*(w_s+w_{sd})*a + w_{sec} = 0.90 \text{ kip/ft} \]
  
  \[ w_{L2} = 0.001*(w_{LL}*a) = 1.00 \text{ kip/ft} \]

*From Chapter 2 of ASCE/SEI 7, the required flexural strength is:*

\[ w_{u2} = 1.2*w_{D2} + 1.6*w_{L2} = 2.68 \text{ kip/ft} \]

\[ M_{u2} = \frac{w_{u2}*L^2}{8} = 678 \text{ kip*ft} \]
Determine The Effective Slab Width, $b_e$

The effective width of the concrete slab is the sum of the effective widths to each side of the beam centerline as determined by the minimum value of the three widths set forth in AISC Specification Section I3.1a:

$$b_{e1} = \frac{L}{8} \times 2 = 11.25 \text{ ft}$$

$$b_{e2} = \frac{a}{2} \times 2 = 10.00 \text{ ft}$$

$$b_e = \text{MIN}(b_{e1}; b_{e2}) = 10.00 \text{ ft}$$

Available Flexural Strength

According to AISC Specification Section I3.2a, the nominal flexural strength shall be determined from the plastic stress distribution on the composite section when $h / tw \leq \sqrt{\frac{3.76 E}{Fy}}$

$$\lambda_w = \frac{d}{t_w} = 54.74$$

Web Class = IF($\lambda_w \leq 3.76 \times \sqrt{\frac{E_s}{F_y}}$); "Compact"; "Non-Compact") = Compact

According to AISC Specification Commentary Section I3.2a, the number and strength of steel headed stud anchors will govern the compressive force, $C$, for a partially composite beam. The composite percentage is based on the minimum of the limit states of concrete crushing and steel yielding as follows:

- Concrete crushing:
  $$A_c = \text{Area of concrete slab within effective width. Assume that the deck profile is 50% void and 50% concrete fill.}$$
  $$A_c = (b_e \times 12) \times (t_s - h_r) + 0.5 \times (b_e \times 12 \times h_r) = 720.00 \text{ in}^2$$
  $$C_c = 0.85 \times f'_c \times A_c = 2448 \text{ kips}$$

- Steel yielding:
  $$C_s = A_s \times F_y = 735 \text{ kips}$$

- Shear transfer:
  60% is used as a trial percentage of composite action as follows:
  $$C_1 = \sum Q_n$$
  $$C_1 = 0.6 \times \text{MIN}(C_c; C_s) = 441 \text{ kips}$$
Location of The Plastic Neutral Axis

The plastic neutral axis (PNA) is located by determining the axis above and below which the sum of horizontal forces is equal. This concept assumes the trial PNA location is within the top flange of the beam.

\[ \sum F_{\text{above PNA}} = \sum F_{\text{below PNA}} \]

\[ C_1 + C_{sf1} = T_{s1} \cdot F_y \]

\[ C_{sf1} = x_{f1} \cdot b_f \cdot F_y \]

\[ T_{s1} = (A_s - x_{f1} \cdot b_f) \cdot F_y \]

\[ x_{f1} = \frac{A_s \cdot F_y - C_1}{2 \cdot b_f \cdot F_y} = 0.45 \text{ in} \]

\[ x_f = \text{IF}(x_{f1} \leq t_f; x_{f1}; t_f) = 0.45 \text{ in} \]

\[ C_{sf} = x_f \cdot b_f \cdot F_y = 147 \text{ kips} \]

\[ T_s = (A_s - x_f \cdot b_f) \cdot F_y = 588 \text{ kips} \]

\[ C = T_s - C_{sf} = 441 \text{ kips} \]

Check the percentage of partial composite action:

\[ \alpha = \frac{C}{\text{MIN}(C_c; C_s)} = 0.60 \]

Check_case = IF( \( \alpha < 0.50 \); "Conservative"; "o.k." ) = o.k.

Determine the nominal moment resistance of the composite section following the procedure in Specification Commentary Section I3.2a: (calculating the sum. of moments about the P.N.A.)

\[ M_2 = \Sigma F \cdot Y \]

\[ a_c = \frac{C}{0.85 \cdot f_c^* \cdot b_e \cdot 12} = 1.08 \text{ in} \]

\[ M_{n2} = \frac{1}{12} \cdot (C \cdot (t_s + x_f - a_c/2) + C_{sf} \cdot (x_f/2) + T_s \cdot (d/2 - x_f)) = 763 \text{ kip*ft} \]

Flexural_safety2 = IF(\( M_{n2} \geq M_{u2} \); "Safe"; "Unsafe") = Safe
Chapter 3: Steel Design
Composite Beam Subjected to Bending

Composite deflection: (AISC specs. I3.1 and Eqn. C-13-1)

\[ Y_{ENA} = \frac{(A_s*d/2+(C/F_y)\times(d+(t_s-a_c)/2))\times((A_s+C/F_y))}{5*\frac{W_{L2}}{12}\times(L \times 12)^4} \]

\[ I_{Lb} = \frac{I_x+A_s\times(Y_{ENA}\times d/2+(C/F_y)\times(d+(t_s-a_c)-Y_{ENA})^2}{384*E_s*I_{lb}} \]

\[ \Delta_c = \frac{\frac{w_{L2}}{12}\times(L \times 12)^4}{384*E_s*I_{Lb}} \]

\[ \Delta_{recom} = \frac{L * 12}{360} \]

Deflection_check2 = IF(\(\Delta_c < \Delta_{recom}\); "Safe"; "Unsafe")

Summary

Pre-composite condition:

\[ M_{u1} = \frac{w_{u1} \times L^2}{8} \]

\[ M_{n1} = \frac{\phi_b \times M_p}{E_s} \]

Flexural_safety1 = IF(\(M_{n1} \geq M_{u1}\); "Safe"; "Unsafe")

\[ \Delta_{nc} = \frac{5\times\frac{w_{D1}}{12}\times(L \times 12)^4}{384*E_s*I_x} \]

\[ \Delta_{recom} = \frac{L * 12}{360} \]

Deflection_check = IF(\(\Delta_{nc} - \text{Camber} < \Delta_{recom}\); "Safe"; "Unsafe")

Composite condition:

\[ M_{u2} = \frac{w_{u2} \times L^2}{8} \]

\[ M_{n2} = \frac{1/12*(C*(t_l+x_f-a_c/2)+C_{sf}*(x_f/2)+T_s*(d/2-x_f))}{E_s} \]

Flexural_safety2 = IF(\(M_{n2} \geq M_{u2}\); "Safe"; "Unsafe")

\[ \Delta_{nc} = \frac{5\times\frac{w_{D1}}{12}\times(L \times 12)^4}{384*E_s*I_x} \]

Deflection_check = IF(\(\Delta_{nc} - \text{Camber} < \Delta_{recom}\); "Safe"; "Unsafe")
Chapter 4: Connection Design

Design of Base Plate Bearing on Concrete Subjected to Concentric Loading

Loads

Dead Load, \( P_D = 115.0 \text{ kips} \)
Live Load, \( P_L = 345.0 \text{ kips} \)

Material Properties

Grade: \( \text{SEL("Material/ASTM"; NAME; )} = \text{A36} \)
Yield stress, \( f_{yp} = \text{TAB("Material/ASTM";Fy;NAME=Grade)} = 36 \text{ ksi} \)

Column and Pedestal Data

Concrete strength for pedestal (\( f'_c \)):
\[ f'_c = 3 \text{ ksi} \]

Sec.: \( \text{SEL("AISC/W"; NAME; )} = \text{W12X96} \)
Pedestal depth, \( P_d = 24 \text{ in} \)
Pedestal width, \( P_w = 24 \text{ in} \)
Depth of column, \( d = \text{TAB("AISC/W";d; NAME=Sec.)} = 12.7 \text{ in} \)
Flange of column, \( b_f = \text{TAB("AISC/W";b_f;NAME=Sec.)} = 12.2 \text{ in} \)

The Required Strength

(Chapter 2 of ASCE/SEI 7)
\[ P_u = 1.2 \times P_D + 1.6 \times P_L = 690 \text{ kips} \]

Preliminary Base Plate Dimensions

\[ \phi_c = 0.65 \]
\[ A_{treq} = \frac{P_u}{\phi_c \times 0.85 \times f'_c} = 416 \text{ in}^2 \]
\[ N1 = d + 2 \times 3 = 18.7 \text{ in} \]
\[ N = \text{MAX}(N1;\sqrt{(A_{treq}+0.5^*(0.95^*d-0.8*b_f))}) = 22 \text{ in} \]
Chapter 4: Connection Design

Base Plate Subjected to Concentric Loading

B1 = \( b_f + 2 \times 3 \) = 18.2 in

B = IF((d-b_f)<1;MAX(B1;A_{1req}/N;N);MAX(B1;A_{1req}/N)) = 22 in

Concrete Bearing Strength

Pedestal area, \( A_2 = P_d \cdot P_w \) = 576 in²

Base plate area, \( A_1 = N \cdot B \) = 484 in²

Check plate area = IF(\( A_1 > A_{1req} \);"o.k.";"Unsafe") = o.k.

\( P_b = \text{MIN}(0.85*f'c*A_1*\sqrt{A_2/A_1} ; 1.7*f'c*A_1) \) = 1346 kips

Concrete bearing strength (\( \Phi_c*P_p \)):

\( \Phi_c*P_p = \Phi_c*P_b \) = 875 kips

Check safety, check = IF(\( \Phi_c*P_p > P_u \);"o.k.";"Unsafe") = o.k.

Base Plate Thickness

\( m = \frac{N - 0.95 \cdot d}{2} \) = 4.97 in

\( n = \frac{B - 0.8 \cdot b_f}{2} \) = 6.12 in

\( n' = \frac{\sqrt{d \cdot b_f}}{4} \) = 3.11 in

\( X = \left(4 \cdot d \cdot \frac{b_f}{(d + b_f)^2}\right) \cdot \left(\frac{P_u}{\Phi_c \cdot P_b}\right) \) = 0.79

\( \lambda = \text{MIN}(2 \cdot \sqrt{X}/(1+\sqrt{1-X}) ; 1.00) \) = 1.00

\( l = \text{MAX}(m ; n; \lambda \cdot n') \) = 6.12 in

\( f_{pu} = \frac{P_u}{N \cdot B} \) = 1.43 ksi

\( t_{min} = l \cdot \sqrt{\frac{2 \cdot f_{pu}}{0.9 \cdot f_{yp}}} \) = 1.82 in

\( t = \text{TAB("Material/plate_th";t_fr;t_in>t_{min})} \) = 2.00 in

Summary: Use Plate with The Following Dimensions

Plate length = N = 22 in
Plate width = B = 22 in
Plate thickness = t = 2.00 in
Chapter 4: Connection Design
Base Plate Subjected to Small Eccentricity

Design of Base Plate Bearing on Concrete Subjected to Small Eccentricity, \( e \leq N/6 \)

\[ \text{Loads} \]
- Dead Load, \( P_D = 50 \text{ kips} \)
- Live Load, \( P_L = 90 \text{ kips} \)
- Moment from D.L., \( M_D = 100 \text{ kip}\cdot\text{in} \)
- Moment from L.L., \( M_L = 180 \text{ kip}\cdot\text{in} \)

\[ \text{Base Plate Material Properties} \]
- Grade: \( \text{SEL("Material/ASTM"; NAME; ) = A36} \)
- Yield stress, \( f_{yp} = \text{TAB("Material/ASTM";F_y;NAME=Grade)} = 36 \text{ ksi} \)

\[ \text{Column, Base Plate and Pedestal Dimensions} \]
- Concrete strength for pedestal (\( f'_{c} \)):
  \[ f'_{c} = 3 \text{ ksi} \]
- Sec.: \( \text{SEL("AISC/W"; NAME; ) = W10X112} \)
- Pedestal depth, \( P_d = 17 \text{ in} \)
- Pedestal width, \( P_w = 14 \text{ in} \)
- Base plate depth, \( N = 17 \text{ in} \)
- Base plate width, \( B = 14 \text{ in} \)
- Depth of column, \( d = \text{TAB("AISC/W";d; NAME=Sec.) = 11.4 \text{ in}} \)
- Flange of column, \( b_f = \text{TAB("AISC/W";b_f;NAME=Sec.) = 10.4 \text{ in}} \)

\[ \text{Check Eccentricity Size} \]
- \( M_t = M_D + M_L = 280 \text{ kip}\cdot\text{in} \)
- \( P_t = P_D + P_L = 140 \text{ kips} \)
- \( e = \frac{M_t}{P_t} = 2.00 \text{ in} \)
- \( \text{Check}_e = \text{IF}(e \leq N/6; "O.K.", "not O.K." ) = \text{O.K.} \)

(Note that: if this check was not O.K., this template will give a wrong solution)
Chapter 4: Connection Design
Base Plate Subjected to Small Eccentricity

The Ultimate Load and Moment

(Chapter 2 of ASCE/SEI 7)

\[ P_u = 1.2P_D + 1.6P_L \]
\[ M_u = 1.2M_D + 1.6M_L \]

\[ P_u = 204 \text{ kips} \]
\[ M_u = 408 \text{ kip*in} \]

The Maximum Bearing Stress, \( F_b \)

\[ \Phi_c = 0.60 \]
\[ A_1 = NBC \]
\[ A_2 = P_dP_w \]
\[ F_{b1} = 0.85\Phi_c f'_c \sqrt[3]{(A_2/A_1)} \]
\[ F_{b2} = 1.7f'_c \]
\[ F_b = \text{MIN}(F_{b1}; F_{b2}) = 1.53 \text{ ksi} \]

\[ F_1 = \text{IF}(\text{Check}_e=\text{"O.K."}; \frac{P_u}{A_1} \left( \frac{M_u* N/2}{B*N^3/12} \right); \text{""}) = 1.46 \text{ ksi} \]

\[ F_2 = \frac{P_u M_u}{A_1 B*N^3/12} \]
\[ F_2 = 0.25 \text{ ksi} \]

Check \_ F = \text{IF}(F_1 < F_b; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}

Base Plate Thickness:

The critical section is at distance \( x \) from the plate edge, where:

\[ x = \frac{N - 0.95 \times d}{2} = 3.09 \text{ in} \]

The distance from the base center to this section \( (x_1) \) and the stress at this section \( (F_s) \) can be calculated as:

\[ x_1 = \frac{N - x}{2} \]
\[ F_s = \text{IF}(\text{Check}_e=\text{"O.K."}; \frac{P_u}{A_1} + \frac{M_u \times x_1}{B*N^3/12}; \text{""}) = 1.24 \text{ ksi} \]

Interactive Design Aids for Structural Engineers
The factored moment for a 1-in strip at this section ($M_s$) can be calculated as follows:

\[
M_s = \frac{F_s \cdot x^2}{2} + \frac{(F_1 - F_s)\cdot x^2 \cdot 0.67}{2} \quad = \quad 6.62 \text{ kip}\cdot\text{in}
\]

\[
t_{p,min} = \sqrt{\frac{4 \cdot M_s}{0.9 \cdot f_{yp}}} \quad = \quad 0.90 \text{ in}
\]

Summary: Use Plate with The Following Minimum Dimensions

- Plate length= $N$ = 17 in
- Plate width= $B$ = 14 in
- Plate thickness= $t_{p,min}$ = 0.90 in
Design of Base Plate Bearing on Concrete Subjected to Large Eccentricity, e > N/2

Loads

- Dead Load, $P_D =$ 21 kips
- Live Load, $P_L =$ 39 kips
- Moment from D.L., $M_D =$ 171 kip*in
- Moment from L.L., $M_L =$ 309 kip*in

Base Plate Material Properties

- Grade: SEL("Material/ASTM"; NAME; ) = A36
- Yield stress, $f_{yp}$ = TAB("Material/ASTM";Fy;NAME=Grade) = 36 ksi

Column, Base Plate and Pedestal Dimensions

- Concrete strength for pedestal ($f'_c$):
  - $f'_c =$ 3 ksi
- Sec.: SEL("AISC/W"; NAME; ) = W8X35
- Pedestal depth, $P_d =$ 28 in
- Pedestal width, $P_w =$ 28 in
- Base plate depth, $N =$ 14 in
- Base plate width, $B =$ 14 in
- Distance to bolt, $y =$ 1.50 in
- Depth of column, $d =$ TAB("AISC/W";d; NAME=Sec.) = 8.1 in
- Flange of column, $b_f =$ TAB("AISC/W";b_f;NAME=Sec.) = 8.0 in

Check Eccentricity Size

- $M_t =$ $M_D + M_L =$ 480 kip*in
- $P_t =$ $P_D + P_L =$ 60 kips
- $e =$ $M_t/P_t =$ 8.00 in
- Check $e =$ IF($e > N/2$;"O.K.";"not O.K.") = O.K.

(Note that: if this check was not O.K., this template will give a wrong solution)
The Ultimate Load and Moment

(Chapter 2 of ASCE/SEI 7)

\[ P_u = 1.2P_D + 1.6P_L \]
\[ M_u = 1.2M_D + 1.6M_L \]

\[ P_u = 88 \text{ kips} \]
\[ M_u = 700 \text{ kip}^*\text{in} \]

The Maximum Bearing Stress, \( F_b \)

\[ \phi_c = 0.60 \]

\[ A_1 = N\times B = 196.00 \text{ in}^2 \]

\[ A_2 = P_d \times P_w = 784.00 \text{ in}^2 \]

\[ F_{b1} = 0.85\phi_c f'c \sqrt{(A_2/A_1)} = 3.06 \text{ ksi} \]

\[ F_{b2} = 1.7f'c = 5.10 \text{ ksi} \]

\[ F_b = \min(F_{b1}; F_{b2}) = 3.06 \text{ ksi} \]

\[ N' = N - y = 12.50 \text{ in} \]

\[ F' = \frac{F_b \times B \times N'}{2} = 267.8 \text{ ksi} \]

Distance from the base to bolt centers (\( A' \)) is:

\[ A' = \frac{N}{2} - y = 5.50 \text{ in} \]

\[ M_1 = P_u \times A' + M_u = 1184 \text{ kip}^*\text{in} \]

\[ A_1 = \frac{F_b \times B}{3} = 32.4 \text{ in} \]

\[ A_2 = \frac{F_b \times B}{3} = 5.1 \text{ in} \]

\[ A = \min(A_1; A_2) = 5.1 \text{ in} \]

Check_A = IF(A>N';"O.K.";"not O.K.") = O.K.

(If this check was not O.K., increase plate dimensions to find a solution or check inputs)

Tension in Bolts

\[ T = \frac{F_b \times A \times B}{2} - P_u = 21.2 \text{ kips} \]
Base Plate Thickness

The critical section is at distance \( x \) from the plate edge, where:

\[
x = \frac{N - 0.95 \cdot d}{2} = 3.15 \text{ in}
\]

The distance from the zero stress point to this section \( x_1 \) and the stress at this section \( F_s \) can be calculated as:

\[
x_1 = A - x = 1.95 \text{ in}
\]
\[
F_s = \frac{x_1}{A} \cdot F_b = 1.17 \text{ ksi}
\]

The factored moment for a 1-in strip at this section \( M_{s1} \) can be determined from the bearing stress distribution as follows:

\[
M_{s1} = \frac{F_s \cdot x^2}{2} + \frac{(F_b - F_s) \cdot x^2 \cdot 0.67}{2} = 12.09 \text{ kip*in}
\]

Also, the moment based on the critical section on the anchor bolt side is determined as follows (it is assumed that the critical plate width is based on the load spreading out at 45 degrees):

\[
M_{s2} = \frac{T/2(x-y)}{2(x-y)} = 5.30 \text{ kip*in}
\]

So, the critical moment is the maximum from \( M_{s1} \) and \( M_{s2} \) as follows:

\[
M_s = \text{MAX}(M_{s1}; M_{s2}) = 12.09 \text{ kip*in}
\]
\[
t_{p,\text{min}} = \sqrt{\frac{4 \cdot M_s}{0.9 \cdot f_{yp}}} = 1.22 \text{ in}
\]

Summary: Use Plate with the following Minimum Dimensions

- Plate length: \( N = 14 \text{ in} \)
- Plate width: \( B = 14 \text{ in} \)
- Plate thickness: \( t_{p,\text{min}} = 1.22 \text{ in} \)
Design of a Shear Lug for Base Plates Subjected to Axial and Shear Loads

Loads
- Dead Load, $P_D =$ 120 kips
- Live Load, $P_L =$ 150 kips
- Shear Load, $V_w =$ 55 kips

Base Plate Material Properties
- Grade: SEL("Material/ASTM"; NAME = Grade) = A36
- Yield stress, $f_{y_p} =$ TAB("Material/ASTM"; Fy; NAME = Grade) = 36 ksi

Column, Base Plate and Pedestal Dimensions
- Concrete strength for pedestal ($f'_c =$): 3 ksi
- Base plate depth, $N =$ 14 in
- Base plate width, $B =$ 14 in
- The coefficient of friction, $\mu =$ 0.55
- Shear lug width, $W =$ 8 in
- Grout depth, $G =$ 1.0 in

The Portion of The Shear which can be Transferred by Friction Equal to $\mu$:

$V_{lg_u} = 1.3V_w - \mu(0.9P_D) = 12.1$ kips

The Required Bearing Area

$\Phi_c = 0.60$

$A_{lg_u} = \frac{V_{lg_u}}{0.85\Phi_c f'_c} = 7.9$ in$^2$

The Height of The Bearing Portion

$H = \frac{A_{lg_u}}{W} = 0.99$ in
Base Plate Thickness

\[ M_{lg} = \frac{V_{lg} \cdot H + G}{W} \cdot 2 \quad = \quad 1.5 \text{kip}\cdot\text{in} \]

\[ t_{lg} = \sqrt{\frac{4 \cdot M_{lg}}{0.9 \cdot f_{yp}}} \quad = \quad 0.43 \text{in} \]

Summary: Use Shear Lug with the Following Minimum Dimensions

- Depth = H+G = 2 in
- Width = W = 8 in
- Thickness = t_{lg} = 0.43 in
Design of Fillet Weld Subjected to Longitudinal Shear Force

Details of The Connected Plates

Grade: \( \text{SEL("Material/ASTM"; NAME; )} = \text{A992} \)
\( F_y = \text{TAB("Material/ASTM";F\_y;NAME=Grade)} = 50 \text{ ksi} \)
Thickness of the first plate, \( t_{p_1} = 0.2500 \text{ in} \)
Thickness of the second plate, \( t_{p_2} = 0.3750 \text{ in} \)
Width of the perpendicular part, \( w_{pp} = 18.0 \text{ in} \)

Loads

Force: \( \text{SEL ("AISC/force"; type; )} = \text{Tension} \)
Dead load, \( P_D = 33.0 \text{ kips} \)
Live load, \( P_L = 100.0 \text{ kips} \)
From Chapter 2 of ASCE/SEI 7, the required strength is:
\( P_u = 1.2 \times P_D + 1.6 \times P_L = 199.6 \text{ kips} \)

Preliminary Welding Details

Electrode classification number, \( F_{EXX} = 70 \text{ ksi} \)
Length of weld on each side, \( l_w = 27.00 \text{ in} \)
Thickness of weld, \( t_w = 0.1875 \text{ in} \)
Chapter 4: Connection Design

Fillet Weld Subjected to Longitudinal Shear Force

Design of Weld

- Check the maximum and minimum Weld Size (AISC Specification Section J2.2b)

Minimum thickness of the connected parts ($t_{p\text{min}}$):

\[
\begin{align*}
t_{p\text{min}} &= \text{MIN}( t_{p1}; t_{p2}) = 0.2500 \text{ in} \\
t_{w,\text{max}} &= \text{IF}(t_{p\text{min}}<1/4; t_{p\text{min}}; (t_{p\text{min}}-1/16)) = 0.1875 \text{ in} \\
t_{w,\text{min1}} &= \text{IF}(t_{p\text{min}}<1/4; 1/8; \text{IF}((t_{p\text{min}})>1/4 \text{ AND } t_{p\text{min}}<1/2); 3/16; 0)) = 0.1250 \text{ in} \\
t_{w,\text{min2}} &= \text{IF}(t_{w,\text{min1}}>1/2 \text{ AND } t_{w,\text{min}}<3/4); 1/4; 5/16) = 0.3125 \text{ in} \\
t_{w,\text{min}} &= \text{IF}(t_{w,\text{min1}}=0; t_{w,\text{min2}}; \text{MIN}(t_{w,\text{min1}}; t_{w,\text{min2}})) = 0.1250 \text{ in} \\
\text{Check}_1 &= \text{IF}((t_w \geq t_{w,\text{min}} \text{ AND } t_w \leq t_{w,\text{max}}); \text{"O.K."}; \text{"Increase } t_w \text{"}) = \text{O.K.}
\end{align*}
\]

- Minimum required length:

\[
\begin{align*}
l_{w,\text{min}} &= \frac{P_u}{0.60 \cdot F_{\text{EXX}} \cdot t_w / \sqrt{2} \cdot 0.75 \cdot 2} = 23.9 \text{ in}
\end{align*}
\]

- Check length for perpendicular plate width:

\[
\begin{align*}
\text{Check1} &= \text{IF}(F=\text{"C"}; \text{"O.K."}; \text{IF}(l_{w,\text{min}} \geq w_{pp}; \text{"O.K."}; \text{"increase } l_w \text{"})) = \text{O.K.}
\end{align*}
\]

- Calculate the effective weld length:

\[
\begin{align*}
\lambda_w &= \frac{l_w}{t_w} = 144.0 \text{ in} \\
\beta_w &= \text{MIN}(1.2-0.002 \cdot \lambda_w; 1) = 0.91 \text{ in} \\
l_{w,\text{eff}} &= \beta_w \cdot l_w = 24.57 \text{ in}
\end{align*}
\]

- Recheck the weld at its reduced strength:

\[
\begin{align*}
\Phi R_n &= \frac{0.75 \cdot 2 \cdot l_{w,\text{eff}} \cdot t_w / \sqrt{2} \cdot 0.6 \cdot F_{\text{EXX}}}{205.2 \text{ kips}}
\end{align*}
\]

\[
\begin{align*}
\text{Check}_2 &= \text{IF}(\Phi R_n > P_u; \text{"Safe"}; \text{"Unsafe"}) = \text{Safe}
\end{align*}
\]

Design Summary

- Electrode= $F_{\text{EXX}}$ = 70.0 ksi
- Size= $t_w$ = 0.1875 in
- length= $l_w$ = 27.00 in
- $P_u$ = $1.2 \cdot P_D + 1.6 \cdot P_L$ = 199.6 kips
- $\Phi R_n$ = $\frac{0.75 \cdot 2 \cdot l_{w,\text{eff}} \cdot t_w / \sqrt{2} \cdot 0.6 \cdot F_{\text{EXX}}}{205.2 \text{ kips}}$
- Check_1= $\text{IF}(t_w \geq t_{w,\text{min}} \text{ AND } t_w \leq t_{w,\text{max}}); \text{"O.K."}; \text{"Increase } t_w \text{"}) = \text{O.K.}
- Check_2= $\text{IF}(\Phi R_n > P_u; \text{"Safe"}; \text{"Increase weld size/length"}) = \text{Safe}$

Interactive Design Aids for Structural Engineers
Design of Bolts in Bearing Type Connection Subjected to Combined Tension and Shear Forces

Details of Bolt
- Grade: SEL("AISC/ASTM_bolts"; Name; ) = A325
  \[ F_u = \text{TAB}("AISC/ASTM_bolts"; F_u; Name=Grade) = 120 \text{ ksi} \]
- Bolt: SEL("AISC/bolt"; Size; ) = d3/4
  \[ \text{Bolt diameter, } d_b = \text{TAB}("AISC/bolt"; dia; Size=Bolt) = 0.7500 \text{ in} \]
  \[ \text{Area of bolt, } A_b = \text{TAB}("AISC/bolt"; Area; Size=Bolt) = 0.4418 \text{ in}^2 \]

Loads
- Dead:
  - Tension force, \( T_D = 3.50 \text{ kips} \)
  - Shear force, \( V_D = 1.3 \text{ kips} \)
- Live:
  - Tension force, \( T_L = 12.0 \text{ kips} \)
  - Shear force, \( V_L = 4.0 \text{ kips} \)
  From Chapter 2 of ASCE/SEI 7, the required strength is:
  \[ T_u = 1.2*T_D+1.6*T_L = 23.4 \text{ kips} \]
  \[ V_u = 1.2*V_D+1.6*V_L = 8.0 \text{ kips} \]

Check for Shear
- \( \phi = 0.75 \)
  From AISC Specification Table J3.2,
  The available shear strength (\( F_{nv} \)):
  \[ F_{nv} = 0.40*F_u = 48.0 \text{ ksi} \]
  The available shear stress (\( f_{rv} \)):
  \[ f_{rv} = \frac{V_u}{A_b} = 18.1 \text{ ksi} \]
  Check_Shear = if(\( \phi*F_{nv}>f_{rv}; "Safe"; "Increase d_b" \)) = Safe
Check for Tension Accompanied by Shear

From AISC Specification Table J3.2,

The available tensile strength ($F_{nt}$):

$$F_{nt} = 0.75 F_u = 90.0 \text{ ksi}$$

The available tensile strength of a bolt subject to combined tension and shear is as follows, from AISC Spec. Eqn. J3.3a,

$$F_{nt}' = 1.30 \frac{F_{nt}}{f_{rv}} = 71.8 \text{ ksi}$$

The available tension force ($R_n$):

$$R_n = A_b \cdot \text{MIN}(F_{nt}', F_{nt}) = 31.7 \text{ kips}$$

Check Tension:

$$IF(\phi R_n \geq T_u; "Safe"; "Increase d_b") = Safe$$

Design Summary

Size = $d_b = 0.7500 \text{ in}$

$F_u = \text{TAB("AISC/ASTM_bolts";F_u;Name=Grade)} = 120 \text{ ksi}$

$f_{rv} = V_u / A_b = 18.1 \text{ ksi}$

$F_{nv} = 0.40 F_u = 48.0 \text{ ksi}$

Check Shear:

$$IF(\phi F_{nv} \geq f_{rv}; "Safe"; "Increase d_b") = Safe$$

$T_u = 1.2 T_D + 1.6 T_L = 23.4 \text{ kips}$

$R_n = A_b \cdot \text{MIN}(F_{nt}', F_{nt}) = 31.7 \text{ kips}$

Check Tension:

$$IF(\phi R_n \geq T_u; "Safe"; "Increase d_b") = Safe$$
Design of Slip Critical Connection with Short-Slotted Holes Subjected to Shear Force

Details of Bolt (short slots transverse to the load)

Grade: \( \text{SEL("AISC/ASTM_bolts"; Name; )} = \text{A325} \)

\( F_u = \text{TAB("AISC/ASTM_bolts";F_u;Name=Grade)} = 120 \text{ ksi} \)

Bolt: \( \text{SEL("AISC/J3.1"; Size; )} = \text{d3/4 A325} \)

Bolt diameter, \( d_u = \text{TAB("AISC/J3.1";dia;Size=Bolt)} = 0.7500 \text{ in} \)

Area of bolt, \( A_b = \text{TAB("AISC/J3.1";Area;Size=Bolt)} = 0.4418 \text{ in}^4 \)

Class: \( \text{SEL("AISC/Class";class; )} = \text{Class_A} \)

\( T_b = \text{TAB("AISC/J3.1";T_b;Size=Bolt)} = 28 \text{ kips} \)

\( \mu = \text{TAB("AISC/Class2"; mio; cat=Class_sym)} = 0.35 \)

\( T_b \) is the minimum bolt pretension and \( \mu \) is the mean slip coefficient.

\( D_u = 1.13 \)

Number of slip planes, \( n_s = 2 \)

hole factor, \( h_f = 1.00 \)

Loads

Shear force due to dead load, \( V_D = 3.00 \text{ kips} \)

Shear force due to live load, \( V_L = 7.00 \text{ kips} \)

From Chapter 2 of ASCE/SEI 7, the required strength is:

\( V_u = 1.2 \times V_D + 1.6 \times V_L = 14.8 \text{ kips} \)

Check for Slip Resistance

\( \Phi = 1.00 \)

The design slip resistance, \( \Phi R_n = \Phi \times \mu \times D_u \times h_f \times T_b \times n_s = 22.15 \text{ kips} \)

Check_slip = IF(\( \Phi R_n \geq V_u , "Safe" ; "Unsafe" )) = Safe
Check The Limit State of Bolt Shear

\[ \phi_2 = 0.75 \]

From AISC Specification Table J3.2,

The available shear strength, \( F_{nv} = 0.40 \times F_u \) = 48.0 ksi

The actual shear stress, \( f_{rv} = \frac{V_u}{A_b} \) = 33.5 ksi

Check_Shear = IF(\( \phi_2 \times F_{nv} \geq f_{rv} \);"Safe";"Increase \( d_b \)") = Safe

Design Summary

Size = \( d_b \) = 0.7500 in

\( F_u \) = \( \text{TAB("AISC/ASTM_bolts";F_u;Name=Grade)} \) = 120 ksi

\( V_u \) = 1.2*\( V_D \) + 1.6*\( V_L \) = 14.8 kips

The design slip resistance, \( \Phi R_n = \phi \times \mu \times D_u \times h_t \times T_b \times n_s \) = 22.15 kips

Check_slip = IF(\( \Phi R_n \geq V_u \);"Safe";"Unsafe") = Safe

The available shear stress, \( f_{rv} = \frac{V_u}{A_b} \) = 33.5 ksi

The available shear strength, \( F_{nv} = 0.40 \times F_u \) = 48.00 ksi

Check_Shear = IF(\( \phi_2 \times F_{nv} \geq f_{rv} \);"Safe";"Increase \( d_b \)") = Safe
Chapter 5: Design Loads

Calculation of Wind load for Solid Freestanding Walls & Signs as per ASCE/SEI 7-10 Chapters 26 & 29

Dimensions of Solid Freestanding Walls or Solid Freestanding Signs

- Horizontal Dimension of Wall or Sign, L = 75.0 ft
- Mean top height of Wall or Sign, h = 10.0 ft
- Vertical Dimension of Wall or Sign, s = 10.0 ft

Wind load Parameters

- Basic wind speed, V = 105.00 mph
- Wind Directionality Factor (According to Table 26.6-1 of ASCE/SEI 7), \( K_d = 0.85 \)
- The Exposure Category (According to Cl. 26.7.3 of ASCE/SEI 7):
  - Category: SEL("ASCE/Exp", Category; ) = B
- Gust-Effect Factor (According to Cl. 26.9.1 of ASCE/SEI 7), G = 0.85
- Topographic Factor (According to Cl. 26.8.2 of ASCE/SEI 7), \( K_{zt} = 1.00 \)
Chapter 5: Design Loads
Wind load for Solid Freestanding Walls & Signs

Velocity Pressure

According to (Table 26.9-1 of ASCE/SEI 7),
\[ \alpha = \text{TAB("ASCE/Exp", Alph; Category=Category)} = 7.00 \]

According to (Table 26.9-1 of ASCE/SEI 7),
\[ z_g = \text{TAB("ASCE/Exp", zg; Category=Category)} = 1200.00 \]

The Velocity Pressure Exposure Coefficient (According to Table 29.3-1 of ASCE/SEI 7):
\[ K_h = \text{IF}(h<15; 2.01 \times (15/z_g)^{2/\alpha}; 2.01 \times (h/z_g)^{2/\alpha}) = 0.57 \]

Velocity Pressure (According to Cl. 29.3.2 of ASCE/SEI 7),
\[ q_h = 0.00256 \times K_h \times K_{zt} \times K_d \times V^2 = 13.67 \text{ psf} \]

The gross area of the solid freestanding wall or sign, \( A_s = L \times s = 750 \text{ ft}^2 \)

Ratio of solid area to gross area, \( \varepsilon = 1.00 \)

Reduction Factor for Openings, \( RF = 1 - (1 - \varepsilon)^{1.5} = 1.00 \)

Wind Forces - Case A

Force Coefficient (According to Fig. 29.4-1 of ASCE/SEI 7), \( C_{fa} = 1.33 \)

Design wind force (According to Cl. 29.4.1 of ASCE/SEI 7),
\[ F_A = q_h \times G \times C_{fa} \times A_s \times RF/1000 = 11.6 \text{ kips} \]

Wind Forces - Case B

Force Coefficient (According to Fig. 29.4-1 of ASCE/SEI 7), \( C_{fb} = 1.33 \)

Design wind force (According to Cl. 29.4.1 of ASCE/SEI 7),
\[ F_B = q_h \times G \times C_{fb} \times A_s \times RF/1000 = 11.6 \text{ kips} \]

Wind Forces - Case C

Region-1 from (0 to s or less)
Force Coefficient for Region-1 (According to Fig. 29.4.1 of ASCE/SEI 7),
\[ C_{ic1} = 3.48 \]

Effective Area for Region-1, \( A_{sc1} = \text{IF}(L/s>1; s \times s; L \times s; ) = 100 \text{ ft}^2 \)

Design Wind Force for Region-1 (According to Cl. 29.4.1 of ASCE/SEI 7),
\[ F_{C1} = \text{RF/1000*MAX}(16; q_h \times G \times C_{ic1} \times \text{IF}(s/h>0.8; (1.8-s/h); 1)) \times A_{sc1} = 3.2 \text{ kips} \]

Region-2 from (s to 2s)

Validation: \( \text{IF}(L/s>1; "Valid"; "Invalid"; ) = Valid \)

Force Coefficient for Region-2 (According to Fig. 29.4-1 of ASCE/SEI 7),
\[ C_{ic2} = 2.28 \]

Effective Area for Region-2, \( A_{sc2} = \text{IF}(L/s>2; s \times s; IF(L/s<1; (L-s) \times s; ) = 100 \text{ ft}^2 \)

Design Wind Force for Region-2 (According to Cl. 29.4.1 of ASCE/SEI 7),
\[ F_{C2} = \text{RF/1000*MAX}(16; q_h \times G \times C_{ic2} \times \text{IF}(s/h>0.8; (1.8-s/h); 1)) \times A_{sc2} = 2.1 \text{ kips} \]
Region-3 from (2s to 3s)                    Validation: IF(L/s>2;"Valid","Invalid"); ) = Valid
Force Coefficient for Region-3 (According to Fig. 29.4-1 of ASCE/SEI 7),
\[ C_{fC3} = 1.68 \]
Effective Area for Region-3, \[ A_{sC3} = \begin{cases} \text{L/s>3;0) s; & \text{IF(L/s<2;0;L-2*s)*s;)} \end{cases} \] = 100 ft²
Design Wind Force for Region-3 (According to Cl. 29.4.1 of ASCE/SEI 7).
\[ F_{C3} = RF/1000*MAX(16;q_h*G*C_{fC3}*IF(s/h>0.8;(1.8-s/h);1))*A_{sC3} = 1.6 \text{ kips} \]
Region-4 from (3s to L)                     Validation: IF(L/s>3;"Valid","Invalid"); ) = Valid
Force Coefficient for Region-4 (According to Fig. 29.4-1 of ASCE/SEI 7),
\[ C_{fC4} = 1.05 \]
Effective Area for Region-4, \[ A_{sC4} = IF(L/s>3;L-3*s)*s;0; ) \] = 450 ft²
Design Wind Force for Region-4 (According to Cl. 29.4.1 of ASCE/SEI 7).
\[ F_{C4} = RF/1000*MAX(16;q_h*G*C_{fC4}*IF(s/h>0.8;(1.8-s/h);1))*A_{sC4} = 7.2 \text{ kips} \]

Calculation Summary
\[ F_A = F_A = 11.6 \text{ kips} \]
\[ F_B = F_B = 11.6 \text{ kips} \]
\[ F_{C1} = F_{C1} = 3.2 \text{ kips} \]
\[ F_{C2} = F_{C2} = 2.1 \text{ kips} \]
\[ F_{C3} = F_{C3} = 1.6 \text{ kips} \]
\[ F_{C4} = F_{C4} = 7.2 \text{ kips} \]
## Calculation of Snow Loads for Flat Roof as per ASCE/SEI 7-10 Chapter 7

### Parameters of Snow Load

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value or Expression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Snow Loads (According to Table 7-1 of ASCE/SEI 7), $p_g$</td>
<td>18.00 psf</td>
</tr>
<tr>
<td>Snow Density (According to Eq. 7.7-1 of ASCE/SEI 7), $\gamma$</td>
<td>$\text{MIN}(0.13 \times p_g + 14; 30) = 16.34 \text{ lb/ft}^3$</td>
</tr>
<tr>
<td>Terrain Category (According to Table 7-2 of ASCE/SEI 7), TER_CAT</td>
<td>SEL(&quot;ASCE/Ter_Cat&quot;; ID; ) = B</td>
</tr>
<tr>
<td>Exposure of Roof (According to Table 7-2 of ASCE/SEI 7), EX_RF</td>
<td>SEL(&quot;ASCE/EX_RF&quot;; ID; ) = Fully Exposed</td>
</tr>
<tr>
<td>Exposure Factor (According to Table 7-2 of ASCE/SEI 7), $C_e$</td>
<td>0.90</td>
</tr>
<tr>
<td>Thermal Factor (According to Table 7-3 of ASCE/SEI 7), $C_t$</td>
<td>SEL(&quot;ASCE/Ct&quot;; ID; ) = 1.00</td>
</tr>
<tr>
<td>Risk Category (According to Table 1.5-1 of ASCE/SEI 7), RI_CAT</td>
<td>SEL(&quot;ASCE/Risk_Cat&quot;; ID; ) = II</td>
</tr>
<tr>
<td>Importance Factor (According to Table 1.5-2 of ASCE/SEI 7), $I_s$</td>
<td>TAB(&quot;ASCE/Is&quot;; Is; RI_CAT=RI_CAT; ) = 1.00</td>
</tr>
</tbody>
</table>

### Snow Load for Flat Roof

- **Min Snow Load (According to Cl. 7.3.4 of ASCE/SEI 7),**
  
  $p_m = \text{IF}(p_g>20; 20*I_s; p_g*I_s) = 18.00 \text{ psf}$

- **Flat Roof Snow Load (According to Cl. 7.3 of ASCE/SEI 7),**
  
  $p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 11.34 \text{ psf}$

### Calculation Summary

- **Min Snow Load,** $p_m = p_m = 18.00 \text{ psf}$
- **Flat Roof Snow Load,** $p_f = p_f = 11.34 \text{ psf}$
Calculation of Snow Loads for Sloped Roof as per ASCE/SEI 7-10 Chapter 7

Parameters of Snow Load

Ground Snow Loads (According to Table 7-1 of ASCE/SEI 7), \( p_g = 40.00 \text{ pf} \)

Density of Snow (According to Eq. 7.7-1 of ASCE/SEI 7),
\[
\gamma = \min(0.13 \cdot p_g + 14; 30) = 19.20 \text{ lb/ft}^3
\]

Terrain Category (According to Table 7-2 of ASCE/SEI 7),
\[
\text{TER\_CAT} = \text{SEL}(\"ASCE/Ter\_Cat\"; \text{ID}; ) = \text{B}
\]

Exposure of Roof (According to Table 7-2 of ASCE/SEI 7),
\[
\text{EX\_RF} = \text{SEL}(\"ASCE/EX\_RF\"; \text{ID}; ) = \text{Fully Exposed}
\]

Exposure Factor (According to Table 7-2 of ASCE/SEI 7),
\[
C_e = 0.70
\]

Thermal Factor (According to Table 7-3 of ASCE/SEI 7),
\[
C_t = \text{SEL}(\"ASCE/Ct\"; \text{ID}; ) = 1.10
\]

Risk Category (According to Table 1.5-1 of ASCE/SEI 7),
\[
\text{RI\_CAT} = \text{SEL}(\"ASCE/Risk\_Cat\"; \text{ID}; ) = \text{II}
\]

Importance Factor (According to Table 1.5-2 of ASCE/SEI 7),
\[
I_s = \text{TAB}(\"ASCE/Is\"; Is; \text{RI\_CAT}=\text{RI\_CAT}; ) = 1.00
\]

Snow Load for Flat Roof

Min Snow Load (According to Cl. 7.3.4 of ASCE/SEI 7),
\[
p_m = \begin{cases} 20.00 \text{ psf} & (p_g > 20; 20 \cdot I_s; p_g \cdot I_s) \\ 20.00 \text{ psf} & \end{cases}
\]

Flat Roof Snow Load (According to Cl. 7.3 of ASCE/SEI 7),
\[
p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g = 21.56 \text{ psf}
\]

Snow Load for Sloped Roof

\[
\alpha = 20.0^0
\]

Thermal Resistance Value \( R = 30.0 \text{ Fh ft}^2/\text{Btu} \)

Roof Slope Factor (According to Fig. (7-2b) of ASCE/SEI 7),
\[
C_s = 1.00
\]

Sloped Roof Snow Load (According to Cl.7.4 of ASCE/SEI 7),
\[
p_s = p_f \cdot C_s = 21.56 \text{ in}
\]

Calculation Summary

Min Snow Load, \( p_m = 20.00 \text{ psf} \)

Flat Roof Snow Load, \( p_f = 21.56 \text{ psf} \)

Sloped Roof Snow Load, \( p_s = 21.56 \text{ in} \)
Chapter 5: Design Loads
Seismic Base Shear

 Calculation of Seismic Base Shear as per ASCE/SEI 7-10 Chapter 11

Site Parameters
(According to Cl.11.4.2 of ASCE/SEI 7),
Site Class: SEL("ASCE/Site_Cl"; ID; ) = A

Mapped Acceleration Parameters
(According to Cl.11.4.1 of ASCE/SEI 7),
At Short Period, $S_s$ = 1.50
At 1 Second Period, $S_1$ = 0.50
Site Coefficient at Short Period, $F_a$ = 1.00
Site Coefficient at 1 Second Period, $F_v$ = 1.30

Spectral Response Acceleration Parameters
(According to Cl.11.4.3 of ASCE/SEI 7),
Spectral Response Acceleration at Short Period, $S_{MS}$ = $F_a * S_s$ = 1.50
Spectral Response Acceleration at 1 Second Period, $S_{M1}$ = $F_v * S_1$ = 0.65

Design Spectral Acceleration Parameters
(According to Cl.11.4.4 of ASCE/SEI 7),
Design Spectral Acceleration Parameter at Short Period, $S_{DS}$ = $2/3 * S_{MS}$ = 1.00
Design Spectral Acceleration Parameter at 1 Second period, $S_{D1}$ = $2/3 * S_{M1}$ = 0.43

Risk Category
Risk Category (According to Table 1.5-1 of ASCE/SEI 7),
$RI\_CAT$ = SEL("ASCE/Risk\_Cat"; ID; ) = I
Importance Factor (According to Table 1.5-12 of ASCE/SEI 7),
$I_e$ = TAB("ASCE/Ie"; Ie; $RI\_CAT$ = $RI\_CAT$) = 1.00

Interactive Design Aids for Structural Engineers
Chapter 5: Design Loads
Seismic Base Shear

Fundamental Period:

(According to Cl.12.8.2 of ASCE/SEI 7),
Type of Structure, \( STR = \) SEL("ACI/Cl&X";STR; ) = All other structural systems
Building Period Parameter, \( C_t = \) TAB("ACI/Cl&X";Ct;STR=STR;) = 0.020
Building Period Parameter, \( x = \) TAB("ACI/Cl&X";X;STR=STR;) = 0.750
Structure Height, \( h_n = \) 66.00 ft

Approximate Fundamental Period, \( T_a = \) \( C_t \cdot h_n^x \) = 0.46 sec
Building fundamental period, \( T = \) \( T_a \) = 0.46 sec
Long-period transition period, \( T_L = \) 2.00

Seismic Response Coefficient

(According to Cl.12.8.1.1 of ASCE/SEI 7)
Response Modification Coefficient (According to Table 12.2-1), \( R = \) 3.25
Calculated Seismic Response Coefficient, \( C_{s,c} = \frac{S_{DS}}{R/I_e} \) = 0.3077
Maximum Seismic Response Coefficient, \( C_{s,max1} = \frac{S_{D1}}{T^* (R/I_e)} \) = 0.2876
Maximum Seismic Response Coefficient, \( C_{s,max2} = \frac{S_{D1} \cdot T_L}{T^2 \times (R/I_e)} \) = 1.2505
Maximum Seismic Response Coefficient, \( C_{s,max} = \) IF(\( T > T_L ; \) \( C_{s,max2} ; \) \( C_{s,max1} \)) = 0.2876
Minimum Seismic Response Coefficient, \( C_{s,min} = \) 0.044 * \( S_{DS} \) * \( I_e \) = 0.0440
Seismic Response Coefficient \( C_s = \) IF(\( C_{s,c} \geq C_{s,min} ; \) MIN(\( C_{s,c} ; \) \( C_{s,max} ; \) \( C_{s,min} \)) = 0.2876

Seismic Base Shear

(According to Cl.12.8.1 of ASCE/SEI 7),
Effective Seismic Weight of Structure, \( W = \) 1305.00 kips
Seismic Base Shear \( V = \) \( C_s \) * \( W \) = 375.32 kips

Calculation Summary

Seismic Base Shear \( V = \) \( C_s \) * \( W \) = 375.32 kips